CHAPTER 6

SHEAR WALLS

6-1. Introduction. This chapter prescribes the criteria for the design of walls of buildings in seismic areas; indicates the principles and factors governing the application of horizontal forces normal to the plane of walls and parallel to the plane of walls; gives certain design data; and illustrates typical details of construction. A wall that carries a vertical load other than its own weight, and/or that resists a horizontal force parallel to the wall, other than seismic shears resulting from its own weight, is classified as a structural wall. Other walls and partitions are classified as nonstructural and are treated in chapter 11.

6-2. General.

a. Function. Shear walls are vertical elements in the lateral force resisting system. They transmit lateral forces from the diaphragm above to the diaphragm below or the foundation.

b. Shear wall types. Two basic shear wall systems are defined in SEAOC Table 1-G; light, framed walls with shear panels of plywood and certain other materials, and shear walls of reinforced concrete and reinforced masonry.

c. Design criteria. General discussions of shear walls are presented in paragraphs 6-3 through 6-6. The details of concrete shear walls are covered in paragraph 6-7, precast concrete shear walls in 6-8, masonry shear walls in 6-9, wood stud shear walls in 6-10, and steel stud shear walls in 6-11.

6-3. Design forces. Walls may be subjected to both vertical (gravity) and horizontal (wind or earthquake) forces. The horizontal forces are both in plane and out of plane. When considered under their in-plane loads, walls are called shear walls; when considered under their out-of-plane loads they are called normal walls. Walls will be designed to withstand all vertical loads and horizontal forces, both parallel to and normal to the flat surface, with due allowance for the effect of any eccentric loading or overturning forces generated. Any wall, whether or not intended as part of the lateral force resisting system, is subjected to lateral forces unless it is isolated on three sides (both ends and top), in which case it is classified as nonstructural. Any wall that is not isolated will participate in shear resistance to horizontal forces parallel to the wall, since it tends to deform under stress when the surrounding framework deforms.

6-4. Wall components. Reinforced concrete and reinforced masonry shear walls are seldom simple walls. Whenever a wall has doors, windows, or other openings, the wall must be considered as an assemblage of relatively flexible components (column segments and wall piers) and relatively stiff elements (wall segments).

a. Column segments. A column segment is a vertical member whose height exceeds three times its thickness and whose width is less than two and one-half times its thickness. Its load is usually predominantly axial. Although it may contribute little to the lateral force resistance of the shear wall, its rigidity must be considered. When a column is built integral with a wall, the portion of the column that projects from the face of the wall is called a pilaster. Column segments shall be designed according to ACI 318 for concrete and TM 5-809-3/AFM 88-3, Chap 3 for masonry.

b. Wall piers. A wall pier is a segment of a wall whose horizontal length is between two and one-half and six times its thickness and whose clear height is at least two times its horizontal length.

c. Wall segments. Wall segments are components that are longer than wall piers. They are the primary resisting components in the shear wall.

6-5. In-plane effects. Horizontal forces at any floor or roof level are generally transferred to the ground (foundation) by using the strength and rigidity of shear walls (and partitions). A shear wall may be considered analogous to a cantilever plate girder standing on end in a vertical plane where the wall performs the function of a plate girder web, the pilasters or floor diaphragms function as web stiffeners, and the integral reinforcement of the vertical boundaries functions as flanges. Axial, flexural, and shear forces must be considered in the design of shear walls. The tensile forces on shear wall elements resulting from the combination of seismic uplift forces and seismic overturning moments must be resisted by anchorage into the foundation medium unless the uplift can be counteracted by gravity loads (e.g., 0.85 of dead load) mobilized from neighboring elements. A shear wall may be constructed of materials such as concrete, wood, unit masonry, or metal in various forms. Design procedures for such materials as cast-in-place reinforced concrete and reinforced unit masonry are well known and present no problem to the designer once the loading and reaction system is determined. Other materials frequently used to support vertical loads from floors and roofs have well-established vertical load carrying characteristics but have required tests to demonstrate their ability to resist lateral forces. Various types of wood sheathing and metal siding fall into this category. Where a shear wall is made up of units such as plywood, gypsum wallboard, tilt-up concrete units, or metal panel units, its characteristics are, to a large degree, dependent upon the attachments of one unit to another and to the supporting members.

a. Rigidity analysis. For a building with rigid diaphragms, there is a torsional moment, and a rigidity analysis is required. Refer to chapter 5. It is necessary to make a logical and consistent distribution of story shears to each wall. An exact determination of wall rigidities is very difficult but is not necessary because only *relative* rigidities are needed. Approximate methods in which the deflections of portions of walls are combined usually are adequate.

(1) *Wall deflections.* The rigidity of a wall is usually defined as the force required to cause a unit deflection. Rigidity is expressed in kips per inch. The deflection of a concrete shear wall is the sum of the shear and flexural deflections. See figure 6-1. In the case of a solid wall with no openings, the computations of deflection are quite simple. However, where the shear wall has openings, as for doors and windows, the computations for deflection and rigidity are much more complex. An exact analysis, considering angular rotation of elements, rib shortening, etc., is very time-consuming. For this reason, several short-cut approximate methods involving more-or-less valid assumptions have been developed. These do not always give consistent or satisfactory results. A conservative approach and judgment must be used.

(2) *Deflection charts.* The calculation of deflections is facilitated by the use of the deflection charts. See figure 6-4 for fixed-ended corner and rectangular piers. Curves 5 and 6 are for cantilever corner and rectangular piers. The corner pier curves are for the special case where the moment of inertia, I, of the corner pier is 1.5 times that of a rectangular pier. For other I-values the bending portion of the deflection would be proportional. The deflections shown on the charts are for a horizontal load, P, of 1,000,000 pounds. The deflections shown on the charts are reasonably accurate. The formulas written on the curves can be used to check the results. However, the charts will give no better results than the assumptions made in the shear wall analysis. For instance, the point of contraflexure of a vertical pier may not be in the center of the pier height. In some cases the point of contraflexure may be selected by judgment and an interpolation made between the cantilever and fixed conditions.

(3) *Foundation effects.* The rotation at the foundation can greatly influence the overall rigidity of a shear wall because of the very rigid nature of the shear wall itself; however, the rotational influence on relative rigidities of walls for purposes of horizontal force distribution may not be as significant. Considering the complexities of soil behavior, a quantitative evaluation of the foundation rotation is generally not practical, but a qualitative evaluation, recognizing the limitations and using good judgment, will be provided.

(4) *Framework effects.* The relative rigidity of concrete or unit masonry walls with openings is

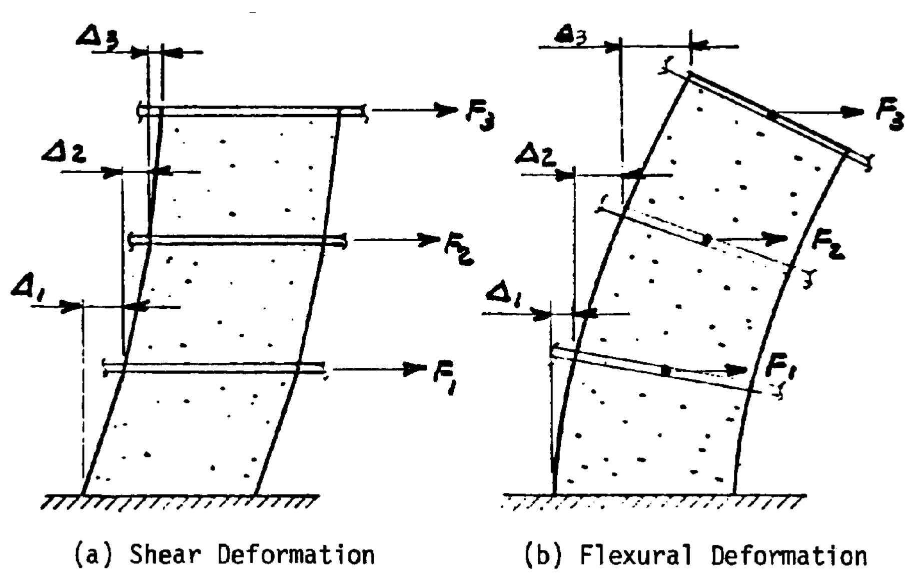


Figure 6-1. Shear wall deformation.

usually much greater than that of the building framework. Thus, the walls tend to resist essentially all or a major part of the lateral force.

b. Effect of openings. The effect of openings on the ability of shear walls to resist lateral forces must be considered. If openings are very small, their effect on the overall state of stress in a shear wall is minor. Large openings have a more pronounced effect and, if large enough, result in a system in which typical frame action predominates. Openings commonly occur in regularly spaced vertical rows throughout the height of the wall, and the connection between the wall sections is provided by either connecting beams (or spandrels), which form a part of the wall, or floor slabs, or a combination of both. If the openings do not line up vertically and/or horizontally, the complexity of the analysis is greatly increased. In most cases, a rigorous analysis of a wall with openings is not required. In the design of a wall with openings, the deformations must be visualized in order to establish some approximate method for analyzing the stress distribution to the wall. Figures 6-1 and 6-2 give some visual descriptions of such deformations. The major points that must be considered are the lengthening and shortening of the extreme sides (boundaries) due to deep beam action, the stress concentration at the corner junctions of the horizontal and vertical components between openings, and the shear and diagonal tension in both the horizontal and vertical components.

(1) *Relative rigidities of piers and spandrels.* The ease of methods of analysis for walls with openings is greatly dependent on the relative rigidities of the piers and the spandrels, as well as the general geometry of the building. Figure 6-3 shows two extreme examples of relative rigidities of exterior walls of a building. In figure 6-3 the piers are very rigid and the spandrels are very flexible. Assuming a rigid base, the shear walls act as vertical cantilevers. When a lateral force is applied, the spandrels act as struts that flexurally deform to be compatible with the deformation of the cantilever piers. It is relatively simple to determine the forces on the cantilever piers by ignoring the deformation characteristics of the spandrels. The spandrels are then designed to be compatible with the pier deformations. In figure 6-3, the piers are relatively flexible compared with the spandrels. The spandrels are assumed to be infinitely rigid, and the piers are analyzed as fixed-ended columns. The spandrels are then designed for the forces induced by the columns. The overall wall system is also analyzed for overturning forces that induce axial forces into the columns. The calculations of relative rigidities for both cases shown in figure 6-3 can be aided by the charts in figure 6-4. For cases of relative spandrel and pier rigidities other than those shown, the analysis and design become more complex.

(2) *Methods of analysis.* Approximate methods for analyzing walls with openings are

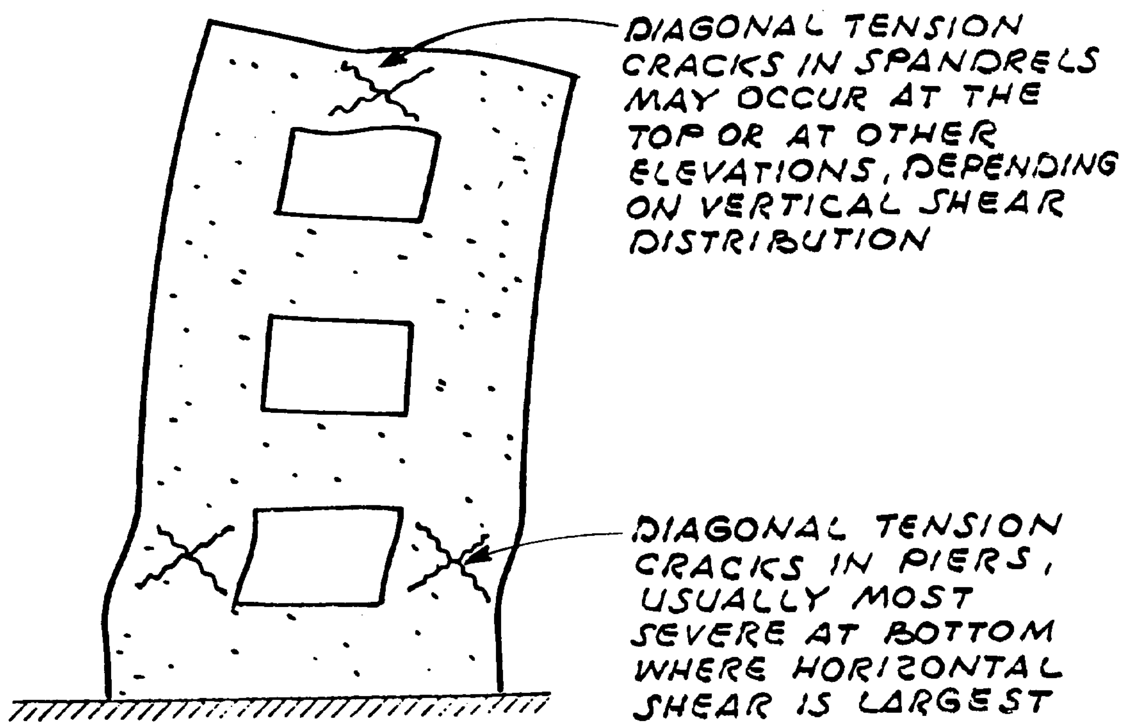


Figure 6-2. Deformation of shear wall with openings.

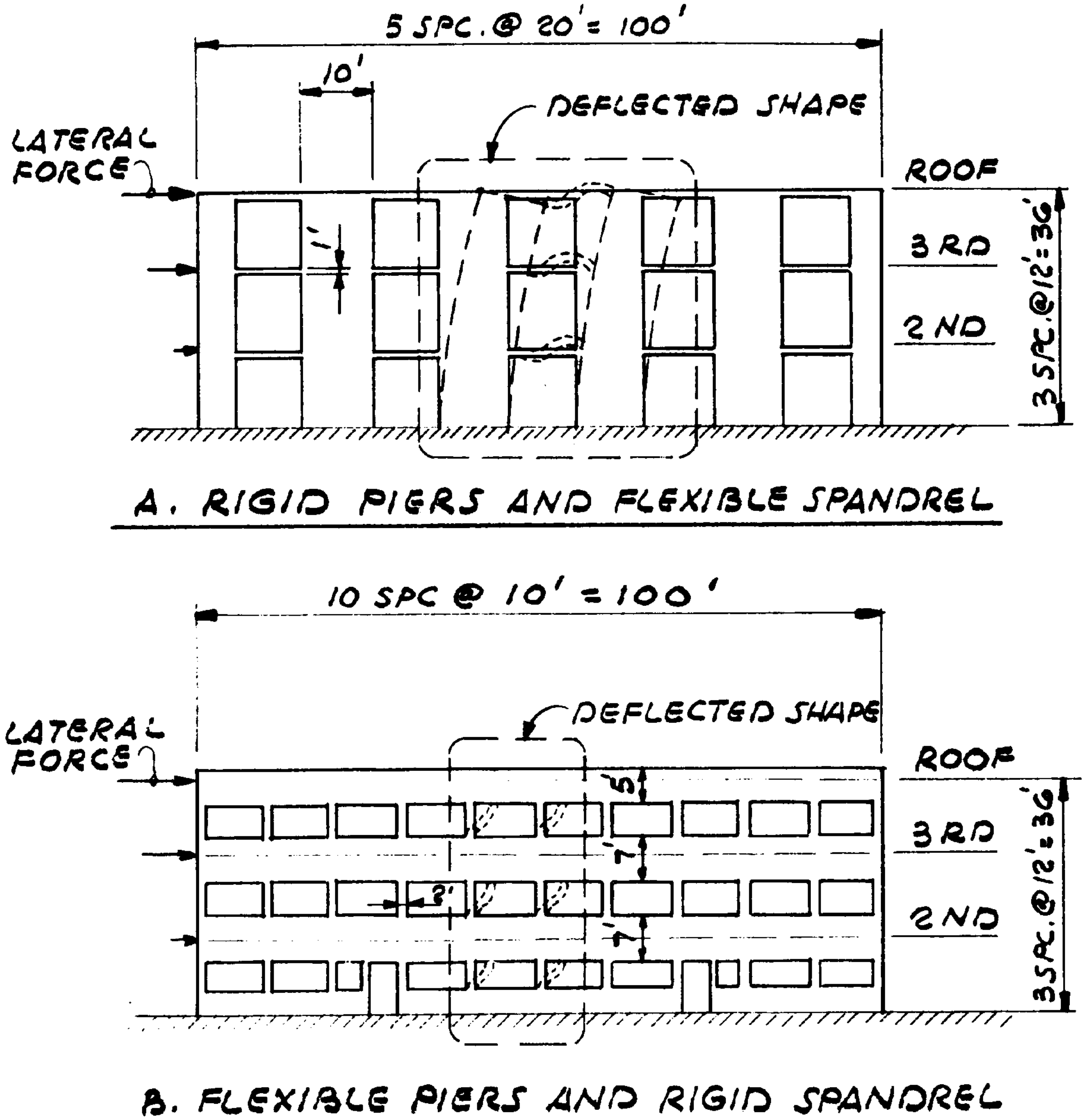


Figure 6-3. Relative rigidities of piers and spandrels.

generally acceptable. For the extreme cases shown in figure 6-3, the procedure is straightforward. For other cases, a variety of assumptions may be used to determine the most critical loads on various elements, thus resulting in a conservative design. (Note: In some cases a few additional reinforcing bars, at little additional cost, can greatly increase the strength of shear walls with openings.) However, when the reinforcement requirements or the resulting stresses of this approach appear excessively large, a rigorous analysis may be justified.

c. Coupled shear walls. When two or more shear walls in one plane are linked together by coupling beams, interactive forces are transmitted to the shear walls by the beams. In addition to these axial forces, the beams develop moments and shears that contribute to the resistance of the walls to overturning. The magnitude of the resisting beam bending moments and vertical shears is dependent on the relative stiffnesses of the walls and the coupling beams. It should be noted that the foundation itself functions as a coupling beam. Accurate determination of the resisting forces can be complex; therefore, approximate methods are generally used. One method may be used for calculating the axial forces and another method may be used for calculating bending moments and shears to ensure that the structural elements are not underdesigned.

d. Construction joints and dowels. The contact faces of shear wall construction joints have exhibited slippage and related drift damage in past earth-

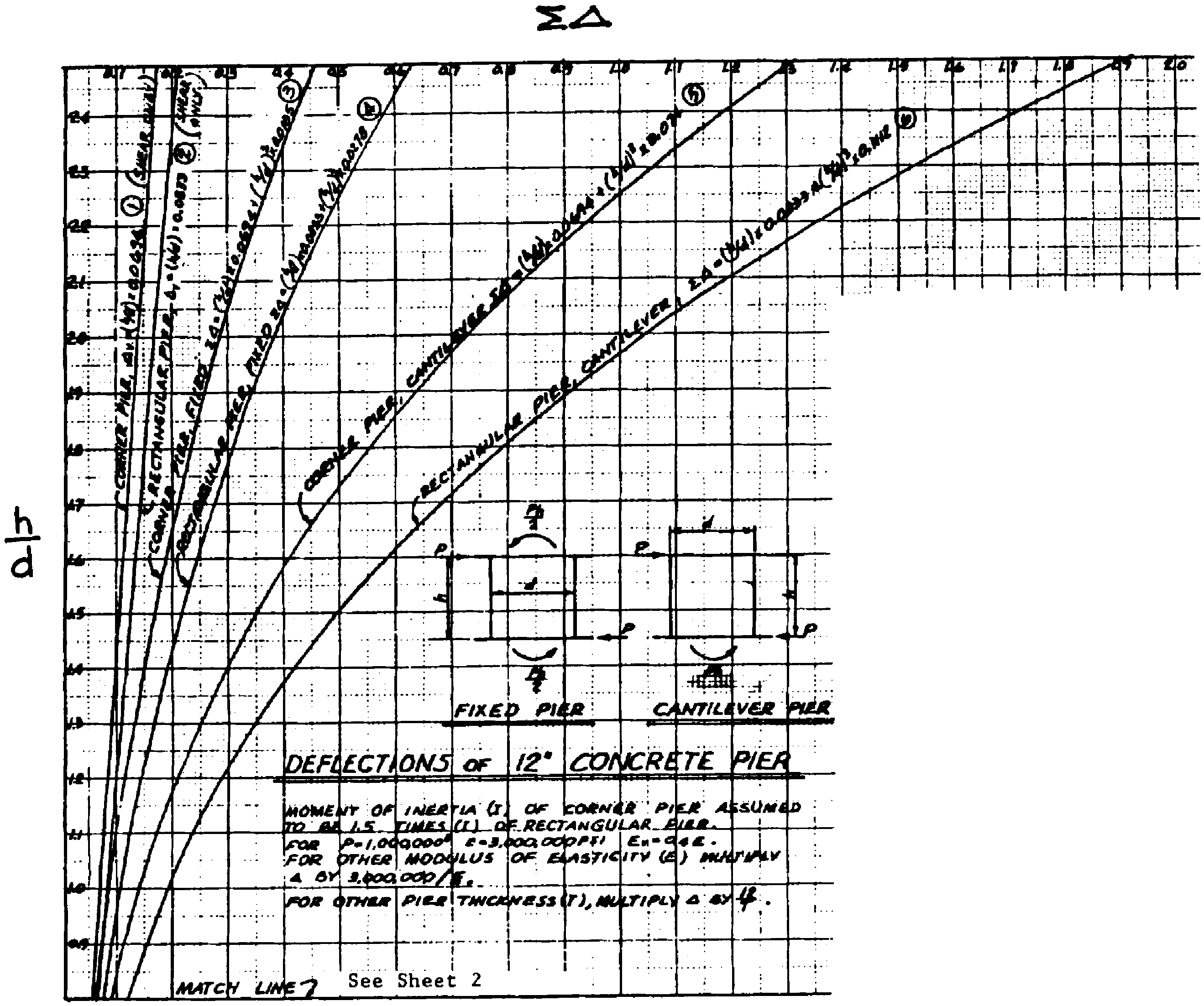


Figure 6-4. Design curves for *masonry and concrete shear walls.*

quakes. Consideration must be given to the location and details of construction joints. They must be clean and roughened. Shear friction reinforcement may be utilized in accordance with ACI 11.7. For this procedure a coefficient of friction of 0.6 is suggested for seismic effects.

6-6. Out-of-plane effects.

a. Lateral forces. Walls and partitions must safely resist horizontal seismic forces normal to their flat surface (fig 6-5, part a). At the same time they must resist moments and shears induced by relative deflections of the diaphragms above and below (fig 6-5, part b). The normal force on a wall is a function of its weight. The equation given in SEAOC 1G is $F_p = ZIC_pW_p$ with $C_p = 0.75$; however, wind forces, other forces, or interstory drift will frequently govern the design. (For cantilevered walls, see paragraph c below.) The design force will be applied to the wall in both inward and outward directions.

b. Wall behavior. Walls usually distribute normal forces vertically to the horizontal resisting elements above or below. They may also distribute normal forces to frames or other walls or frames. A wall may be either continuous or discontinuous across its supports.

c. Cantilevered walls. Where walls, such as parapets, are cantilevered, the anchorage for reaction and cantilever moment is required to be fully developed (fig 6-5, part c). C_p for this condition is

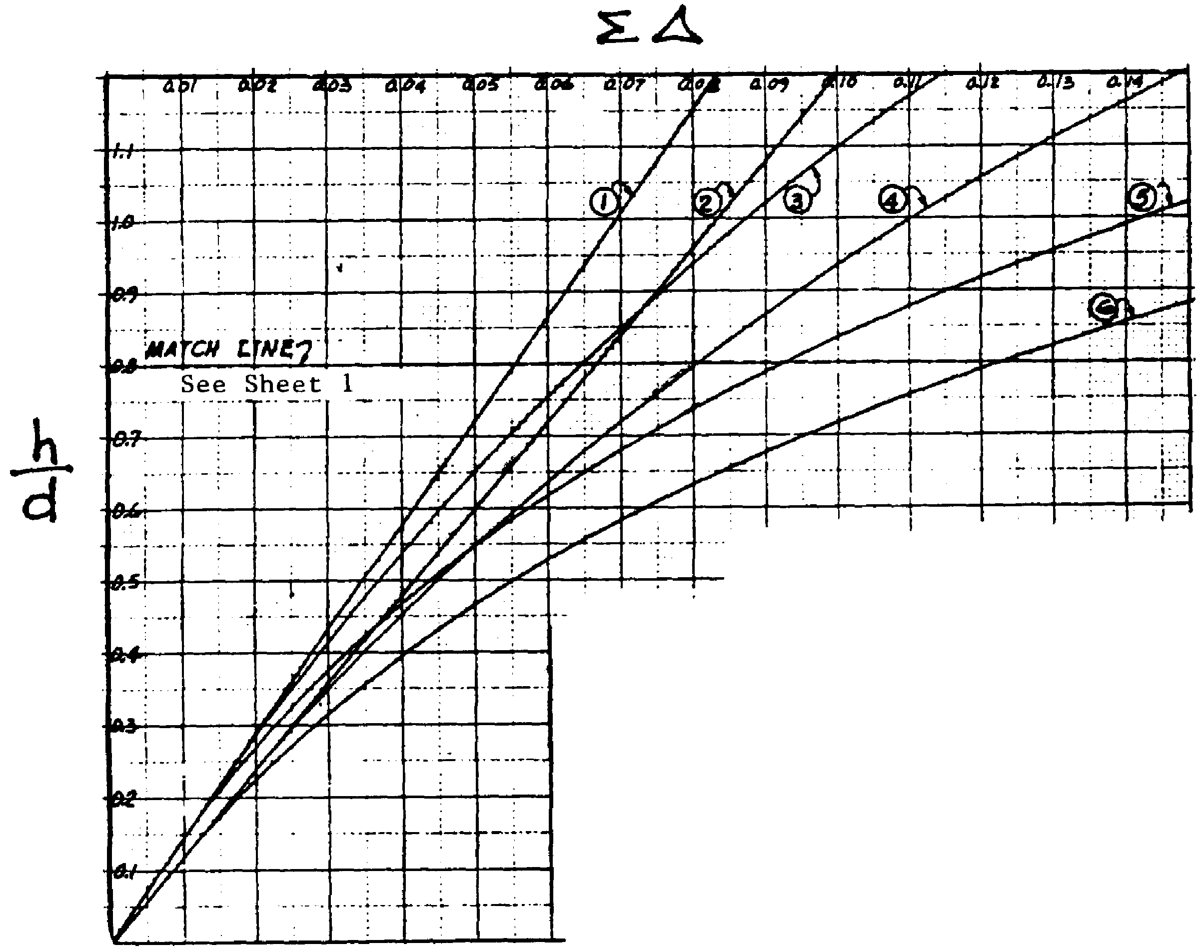


Figure 6-4. Continued

2.00, per SEAOC Table 1-H. Where a parapet wall is anchored to a conrete roof slab and is not a continuation of a wall below, the roof slab will be designed for the cantilever moment. Where the parapet is a continuation of a wall below, the cantilever moment will be divided between the concrete slab and the wall below in proportion to their relative stiffnesses. Where the parapet is an extension of a wall below and is anchored to a roof or floor of wood, metal deck, or other similar materials the moment at the base of the parapet will be developed into the wall below. In this case the anchorage force to the roof will be determined by the usual methods of analysis, assuming a pinned condition for the connection of the roof to the wall.

d. Connections. Walls will be anchored to the structural frame or diaphragm by dowels, anchor bolts, or other approved methods to withstand the design forces, but in no case less than 200 pounds per lineal foot. Dovetail anchors are inadequate for this purpose. Nonstructural partitions will be isolated from exterior walls and shear partitions so as to prevent buttress action, which would restrict shear walls from deflecting with the diaphragms. Isolated partitions will be braced to overhead construction or anchored to other isolated cross walls to ensure lateral stability under out-of-plane loading.

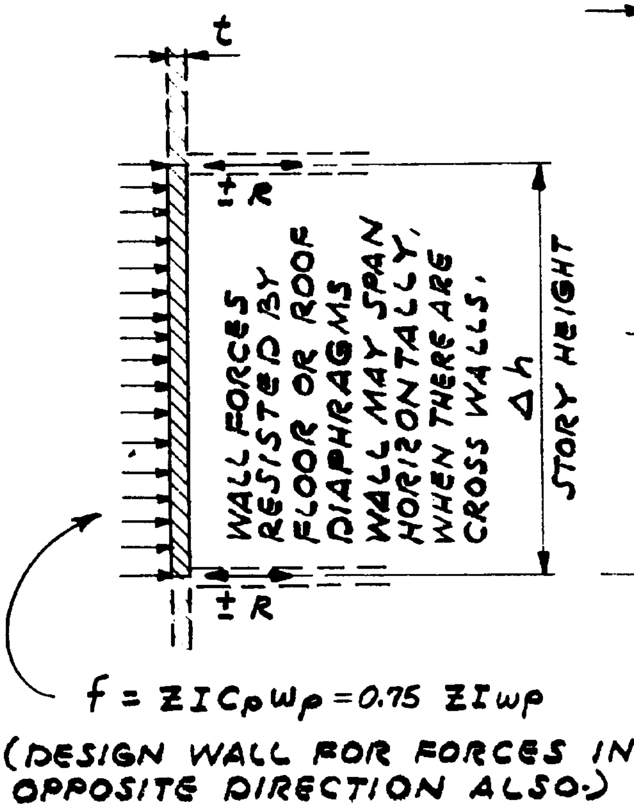
6-7. Cast-in-place concrete shear walls.

a. General design criteria. The criteria used to design reinforced concrete shear walls will be ACI 318 as modified by the amendments given in appendix C. For tilt-up and other precast concrete shear walls, refer to paragraph 6-8. For details of reinforcement, see figures 6-6 and 6-7.

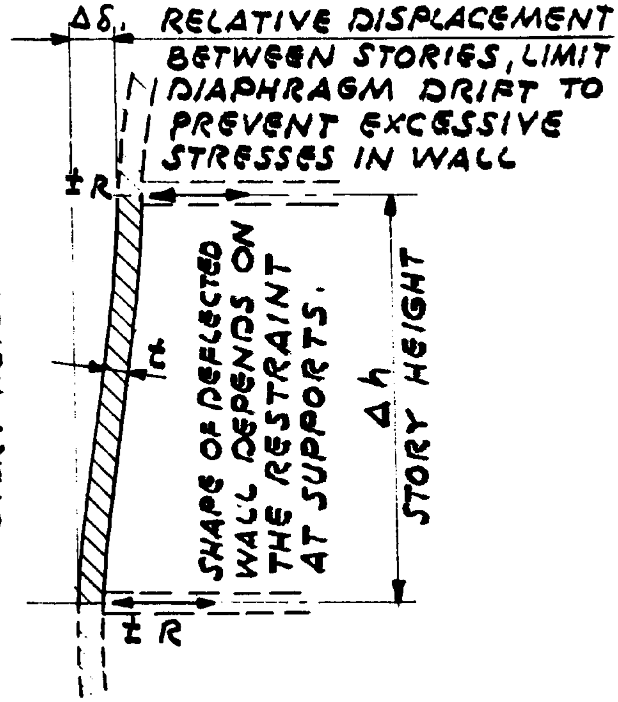
b. Boundary element requirements.

(1) ACI 21.5.3 prescribes when boundary elements are required at the boundaries and edges around openings of shear walls and also specifies that these elements be designed to carry the factored gravity and seismic overturning forces. The elements may be either special reinforced concrete columns (ACI 21.5.3.2) or structural steel columns (SEAOC 3E4).

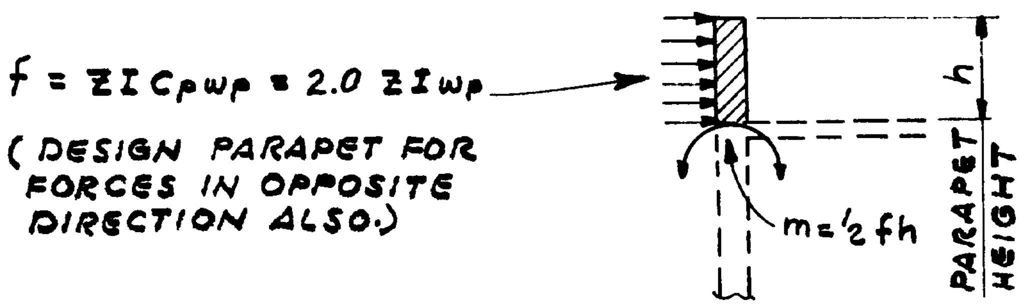
(2) Boundary elements are required when the gross-section compressive stress due to factored



a. Load Normal to Wall



b. Deflections Induced by Relative Deflections of Diaphragms (Refer to Chapter 5)



c. Parapet Loading

Figure 6-5. Out-of -plane effects.

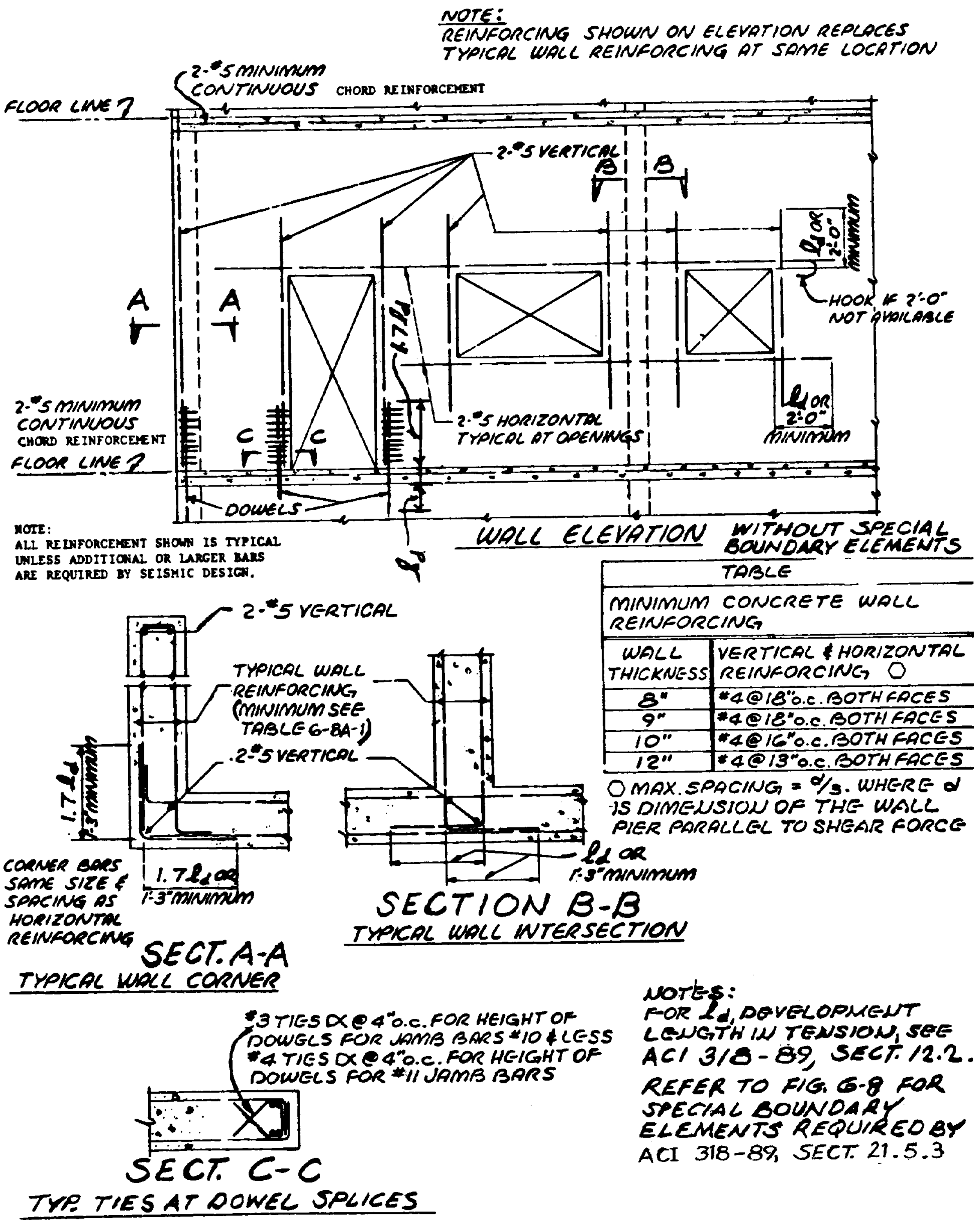


Figure 6-6. Minimum concrete shear wall reinforcement (two curtains).

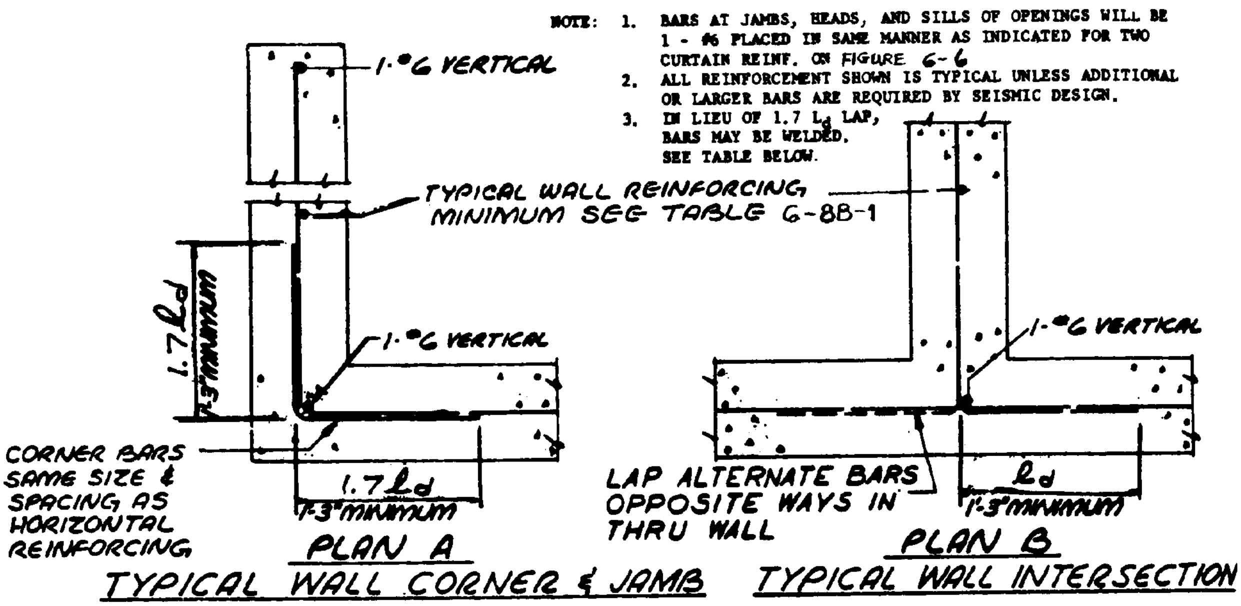


TABLE 6-8B-1

MINIMUM CONCRETE WALL REINF.	
WALL THICKNESS	VERTICAL & HORIZONTAL REINFORCING O
5"	#4 @ 16" o.c. IN CENTER
6"	#4 @ 13" o.c. IN CENTER
7"	#4 @ 11" o.c. IN CENTER
8"	#4 @ 10" o.c. IN CENTER

O SPACING OF BARS NOT TO EXCEED d/3 WHERE d IS DIMENSION OF THE WALL PIER PARALLEL TO SHEAR FORCE

TABLE 6-8B-2

MINIMUM LENGTH OF STANDARD A.W.S. FLARE GROOVE WELDS TO DEVELOP LAPPED REINFORCING BARS	
BAR	WELD LENGTH (EACH SIDE)
3	2"
4	2½"
5	3"
6	3½"
7	4"

Figure 6-7. Minimum concrete shear wall reinforcement (one curtain).

forces exceeds 0.2 f'_c at the edge of the wall boundary or opening.

(3) Wall boundary elements may also occur in the building frame system and the dual system (systems B3a and D1a and b in SEAOC Table 1-G), where the usual configuration is to place the shear walls within the bays between the frame columns. See figure 6-8 for details of shear walls with boundary elements.

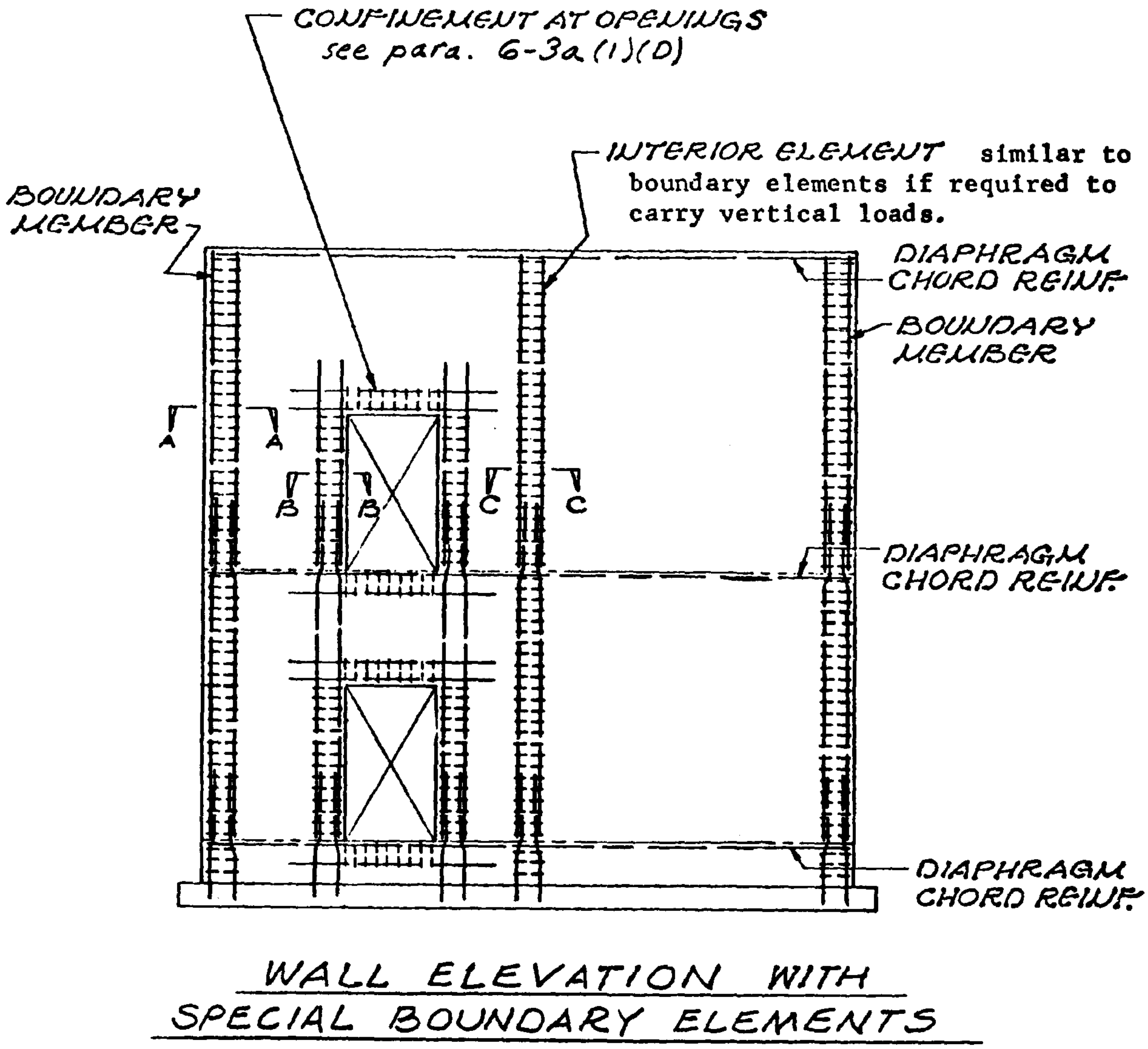
(4) When boundary elements are not required (when gross-section compressive stress is less than or equal to 0.2 f'_c), then only a minimum amount of boundary and edge steel is required by SEAOC 3E2 and 3E3. Details are shown in figures 6-6 and 6-7.

c. Wall piers. Refer to appendix C, paragraph C-19, for design.

6-8. Tilt-up and other precast concrete shear walls.

a. Analysis. Where tilt-up or precast concrete walls are used as shear walls, the analysis is similar to that for walls of cast-in-place concrete; however, in this case the boundary conditions become critical, and the shears between precast and cast-in-place elements must be analyzed. Shears between two precast elements or between a precast element and a cast-in-place element may be developed by shear keys, dowels, or welded inserts. The contact joint itself is a cold joint and will be given no shear or tension value.

b. Joints. Precast concrete elements tend to be structurally separate, one element from another. In the case of precast wall construction, for instance, one might have a series of concrete elements tied

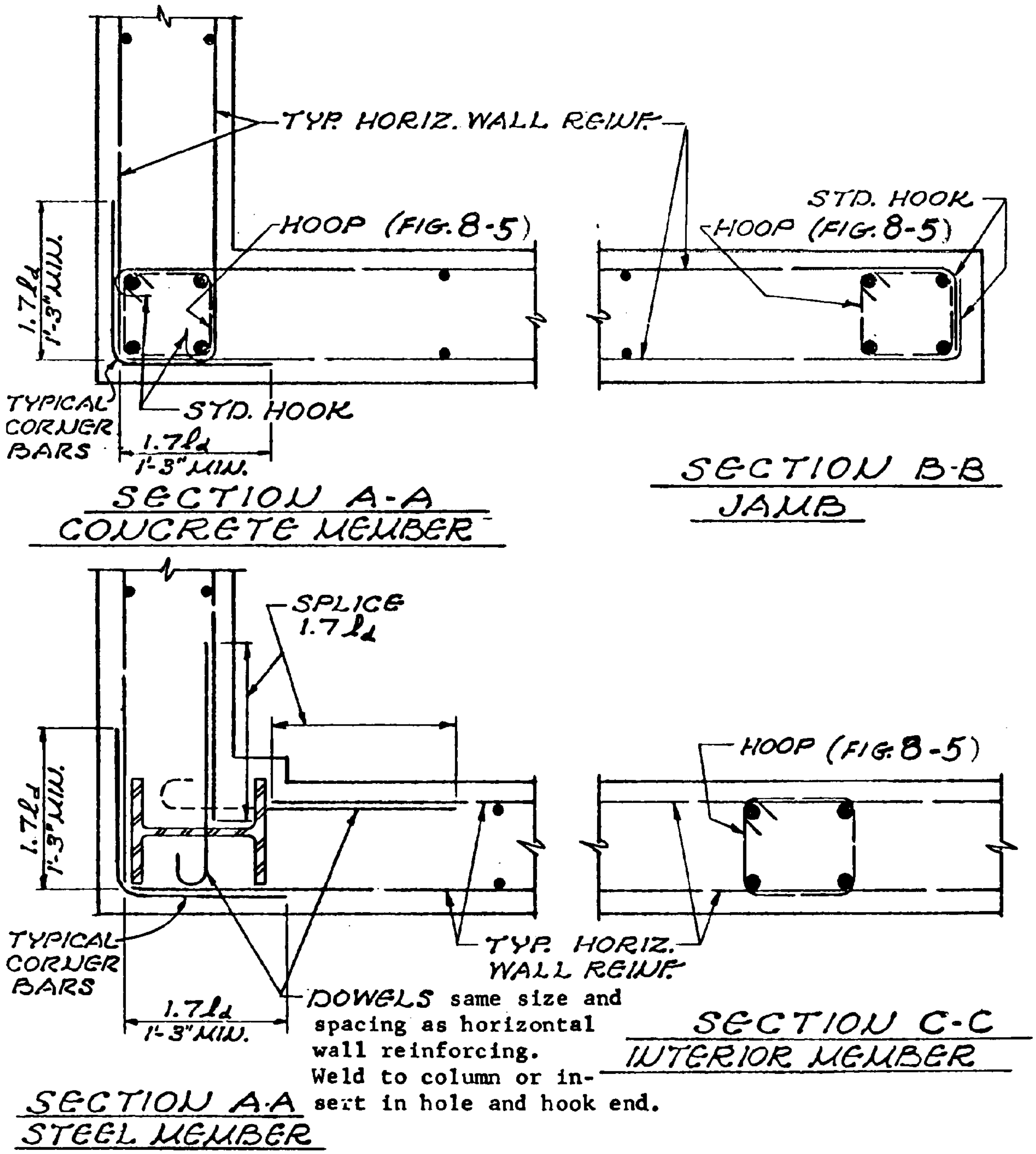


NOTES: For Sections A-A, B-B, and C-C, see Figure 6-8, 2 of 2.
Special vertical boundary members, as shown above, shall be provided at the edges of concrete shear walls in zones 3 and 4 when required by ACI 318-89, Sect. 21.5.3

Figure 6-8. Shear wall with special boundary members.

together at top and bottom but structurally separated from each other by vertical joints. Since all elements in a line are tied together at the top, they must have equal horizontal deflections; therefore, a horizontal force parallel to the line of units will be resisted by the individual elements in proportion to relative rigidities. Such elements may not have equal rigidities, since some may contain large openings or may be of different height-width ratios. Some elements may deflect primarily in shear and others primarily in flexure. Where significant dissimilar deflections are found, the building elements tying the individual units together must be analyzed to determine their ability to resist or accept such deformations, including angular rotation, without losing their ability to function as ties or diaphragm chords or footings. Mechanical keys or sleeved dowels may be used to assist in eliminating differential movement of adjacent precast panels separated by control joints where appearance and weathertightness are otherwise satisfactorily provided.

c. Connectors for shear walls Past earthquakes have shown that the performance of weld plates or other nonductile connectors has often been poor, and in many cases they have resulted in failures.



NOTES: For location of sections, see Figure 6-8, SHEET 1
For l_d, development length in tension, SEE ACI 12.2

Figure 6-8. Continued.

These connectors have been weak links in the shear wall connection. It is important that the load bearing shear walls be more stringently or conservatively designed, since any connector failure during an earthquake may result in progressive failure to collapse. Therefore, all connectors for load-bearing and non-load-bearing walls will be designed for $3(R_w/8)$ times the actual seismic shear forces. The shear force will be uniformly distributed throughout the height or length of the shear wall with reasonably spaced connectors (maximum spacing 4 feet) rather than with a few that will have localized concentration of stresses. Detailed calculations will be made, including the localized effects in concrete walls attributed from these connectors. Sufficient details of connectors and embedded anchorage will be provided to preclude construction deficiency.

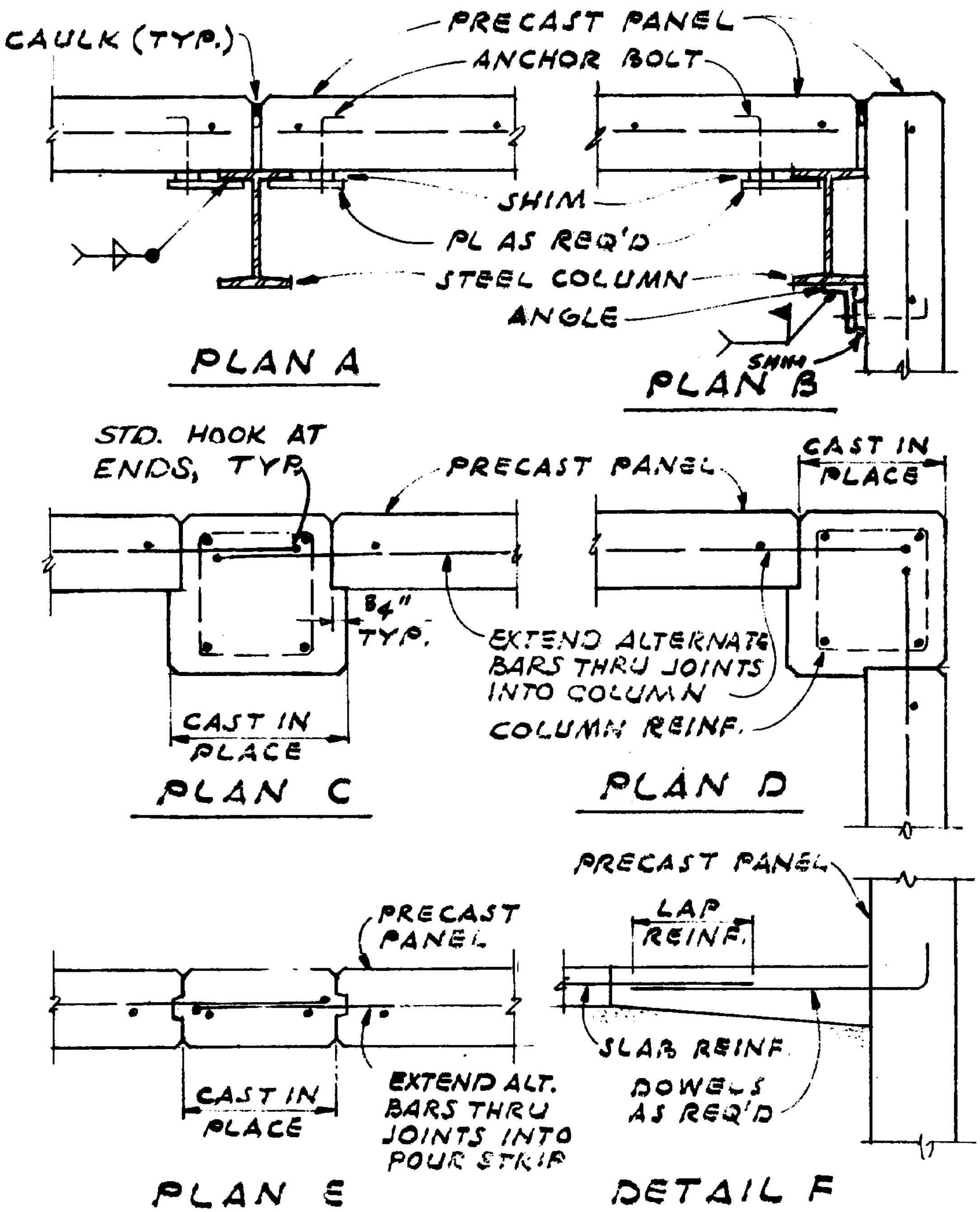


Figure 6-9 Tilt-up and other precast concrete walls-typical details of attachments

d. Typical details. Refer to figure 6-9 for typical details of attachments.

6-9. Masonry shear walls.

a. General design criteria. This section prescribes the criteria for the structural design of shear walls of unit masonry construction. The basic reference document is TM 5-809-3/AFM 88-3, Chap 3. For calculation of shearing stress, masonry shear walls will be designed to resist 1.5 times the forces determined in accordance with SEAOC 2E2.

b. Basic requirements. Unit masonry will be reinforced with deformed bars for axial, flexural, shear, and diagonal tension stresses as determined by design calculations. Additional reinforcing bars are prescribed for use around openings, at corners, at anchored intersections, and at the ends of wall panels (for example, at control joints). The minimum reinforcement prescribed in the manual is intended to provide empirical requirements relative to damage control (ductility and boundary conditions). Layout and details of construction will be compatible with the application of the rules for modular measure.

c. Special requirements.

(1) *Excluded materials.* The following materials will not be used as part of the structural frame:

(a) In Zone 2, glass block, non-load-bearing masonry units, plastic cement, masonry cement, and mortar with more than 1-¼ parts by volume of hydrated lime or lime putty per one part of portland cement.

(b) In Zones 3 and 4, glass block, non-load-bearing masonry units, plastic cement, masonry cement, and mortar with more than ½ part by volume of hydrated lime or lime putty per one part of portland cement.

(2) *Stacked bond.* Since a running bond pattern is the strongest and most economical, the criteria in this manual are based upon each wythe of masonry being constructed in a running bond pattern. The use of a stacked bond pattern will be restricted to reinforced walls essential to the architectural treatment. Filled-cell masonry or grouted masonry will be used. For filled-cell masonry, open end blocks will be used and so arranged that closed ends are not abutting and all head joints are made solid, and bond beam units shall be used to facilitate the flow of grout.

(3) *Height limit.* Unit masonry construction will not be used for shear walls where the height of the building exceeds the limits given in SEAOC Table 1-G.

(4) *Joint reinforcement*. Joint reinforcement will not be used in the calculation of shear strength.

(5) *Mechanical splices.* Mechanical splices will develop 125 percent of the specified yield strength of the bar in tension, except that for compression bars in columns that are not part of the seismic system and are not subject to flexure, the compressive strength only need be developed.

(6) *Cavity walls.* Cavity walls are not practical for use as shear walls because each wythe individually, and both wythes acting together in proportion to their relative rigidities, must be capable of carrying the required loads. It is usually much more economical to construct a two-wythe cavity type wall by using an interior structural wythe and an exterior nonstructural anchored veneer wythe. See chapter 11 for requirements for anchored veneer.

(7) *Drawings.* The locations of control joints and the identification of structural and nonstructural walls and partitions for all masonry construction will be shown on preliminary and contract drawings. On contract drawings, complete details for masonry, reinforcement, and connections to other elements will be shown. Detailing procedures outlined in ACI-315 are generally applicable to reinforced masonry.

(8) *Frame qualifications.* Masonry columns or pilasters will not be used to qualify a structure for a complete vertical load carrying space frame so as to increase the R_w-factor above that of a bearing wall system. Masonry columns will not be used in rigid frame construction.

d. Types of reinforced masonry walls. Masonry will conform to one of the following basic types: reinforced grouted masonry, reinforced hollow masonry, or reinforced filled-cell masonry.

(1) Reinforced grouted masonry is that type of construction made with two wythes of masonry units in which the collar joint between is reinforced and filled solidly with concrete grout. The grout may be placed as the work progresses or after the masonry units are laid. Collar joints will be reinforced with deformed bars, both vertical and horizontal. Reinforcement and embedded items such as structural connections and electrical conduit shall be positioned so as to allow proper placement of grout. All units will be laid in running bond with full shoved head and bed mortar joints. Masonry headers will not project into grout spaces. Clipped-brick headers will be used where the appearance of masonry headers is required. See figure 6-10.

(2) Reinforced hollow masonry is that type of construction made with a single wythe of hollow masonry units (concrete or clay blocks), reinforced vertically and horizontally with steel bars, and cores and voids containing reinforcing bars or embedded items are filled with grout as the work progresses. See figure 6-11.

(3) Reinforced filled-cell masonry is that type of construction made with a single wythe of hollow masonry units, reinforced vertically and horizontally with deformed steel bars, and *all* cores and voids are filled solidly with grout after the wall is laid. See figure 6-12.

e. Bond beams. Bond beams will be located as indicated in figure 6-13. Reinforcement bars in bond beams will be lapped as prescribed in TM 5-809-3/AFM 88-33, Chap 3 at splices, at intersections, and at corners. Bar splices will be staggered. Bond beams will be provided at top of masonry foundation wall stems, below and at top of openings or immediately above lintels, at floor and roof levels, and at top of parapet walls. Intermediate bond beams will be provided as required to conform to the maximum spacing of horizontal bars. However, whenever the height is not a multiple of this normal spacing, the spacing may be increased up to a maximum of 24 inches, provided the bond beams are supplemented with joint reinforcement. One line of joint reinforcement will be provided for each 8-inch increase in the spacing. No additional bond beam will be required between window

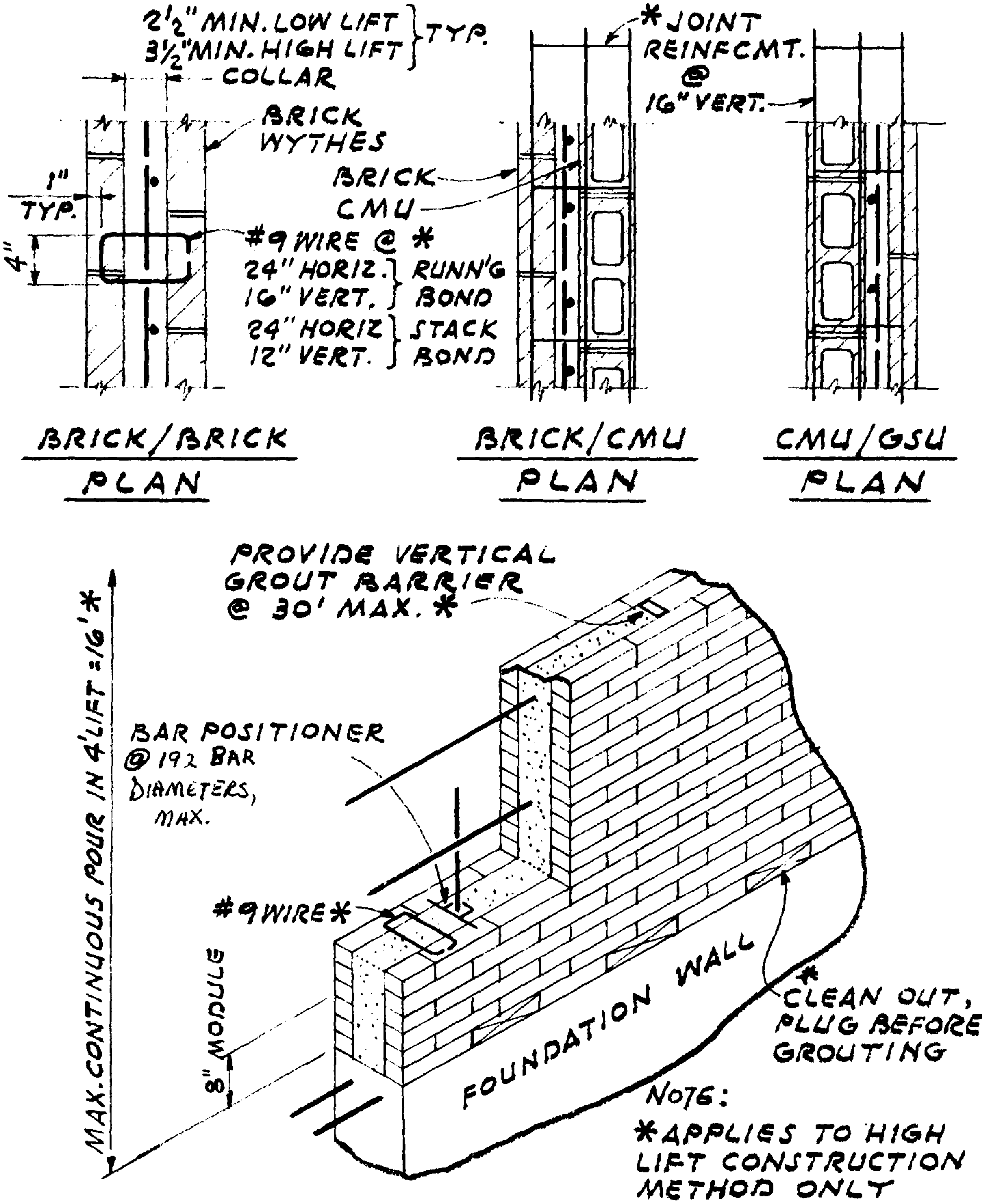


Figure 6-10. Reinforced grouted masonry.

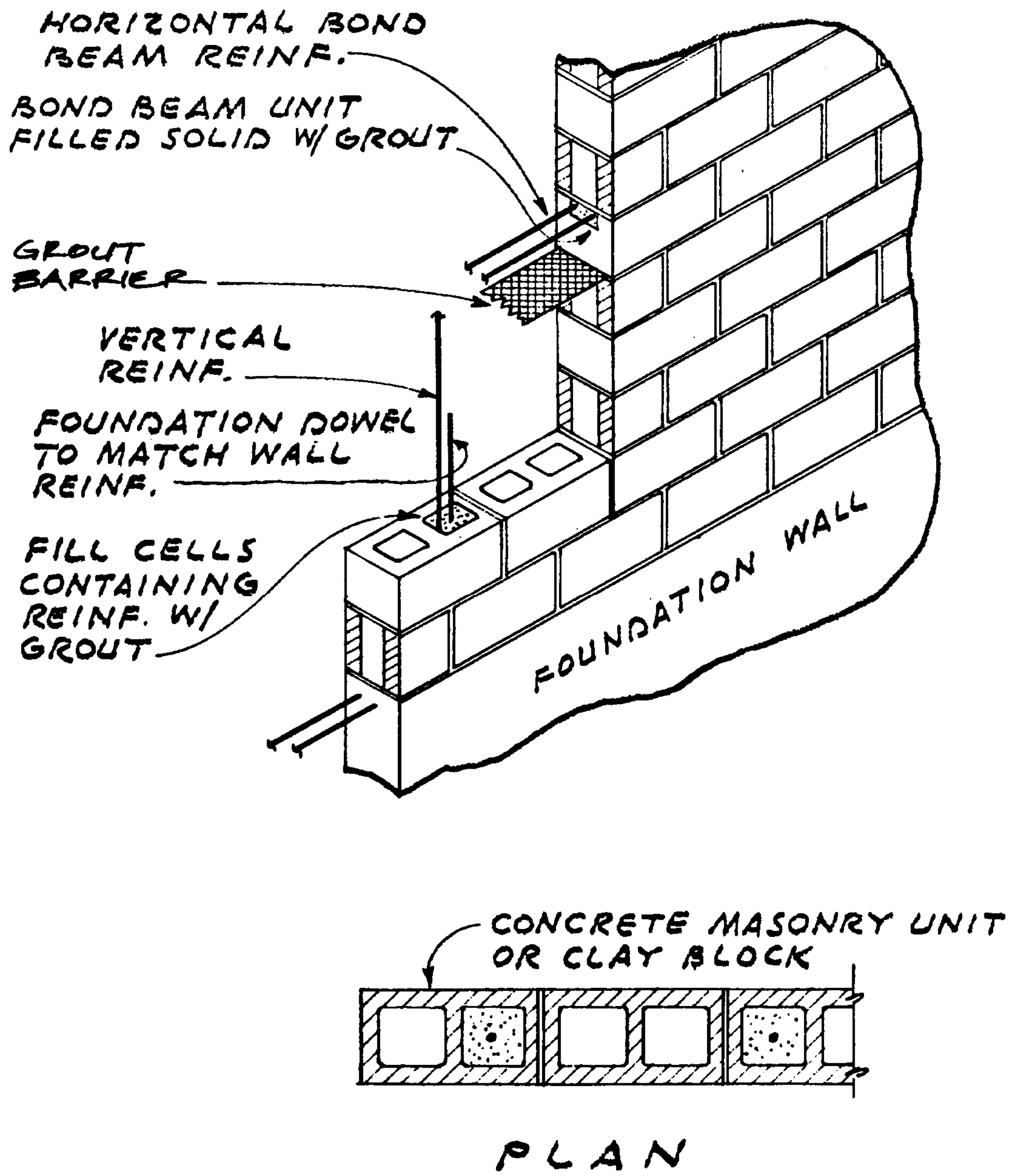


Figure 6-11. Reinforced hollow masonry.

openings that do not exceed 6 feet in height, provided the prescribed supplemental joint reinforcement is installed. To facilitate the placement of steel or concrete core fill, the top bond beam for filler walls or partitions may be placed in the next-to-top course. The area of bond beam reinforcement will be included as part of the minimum horizontal steel.

f. Control joints. Control joints may be required under the provisions of TM 5-809-3/AFM 88-3, Chap 3. The placement of control joints must be coordinated with the seismic design. Because the control joints provide a complete separation of the masonry, the location of control joints fixes the length of wall panels and, in turn, the rigidity of the walls, the distribution of seismic forces, and the resulting unit stresses. Therefore, adding, eliminating, or relocating control joints will not be permitted once the structural design is complete. Control joints will never be assumed to transfer bending moments or diagonal tension across the joint: joint reinforcement and bars in nonstructural bond beams will be terminated at control joints. Deformed bars in structural bond beams (those acting as chords and collectors) will be made continuous for the length of the diaphragm. Refer to figure 6-14.

g. Design considerations.

(1) *Wall weights.* Refer to TM 5-809-3/AFM 88-3, Chap 3 for the average weight of concrete

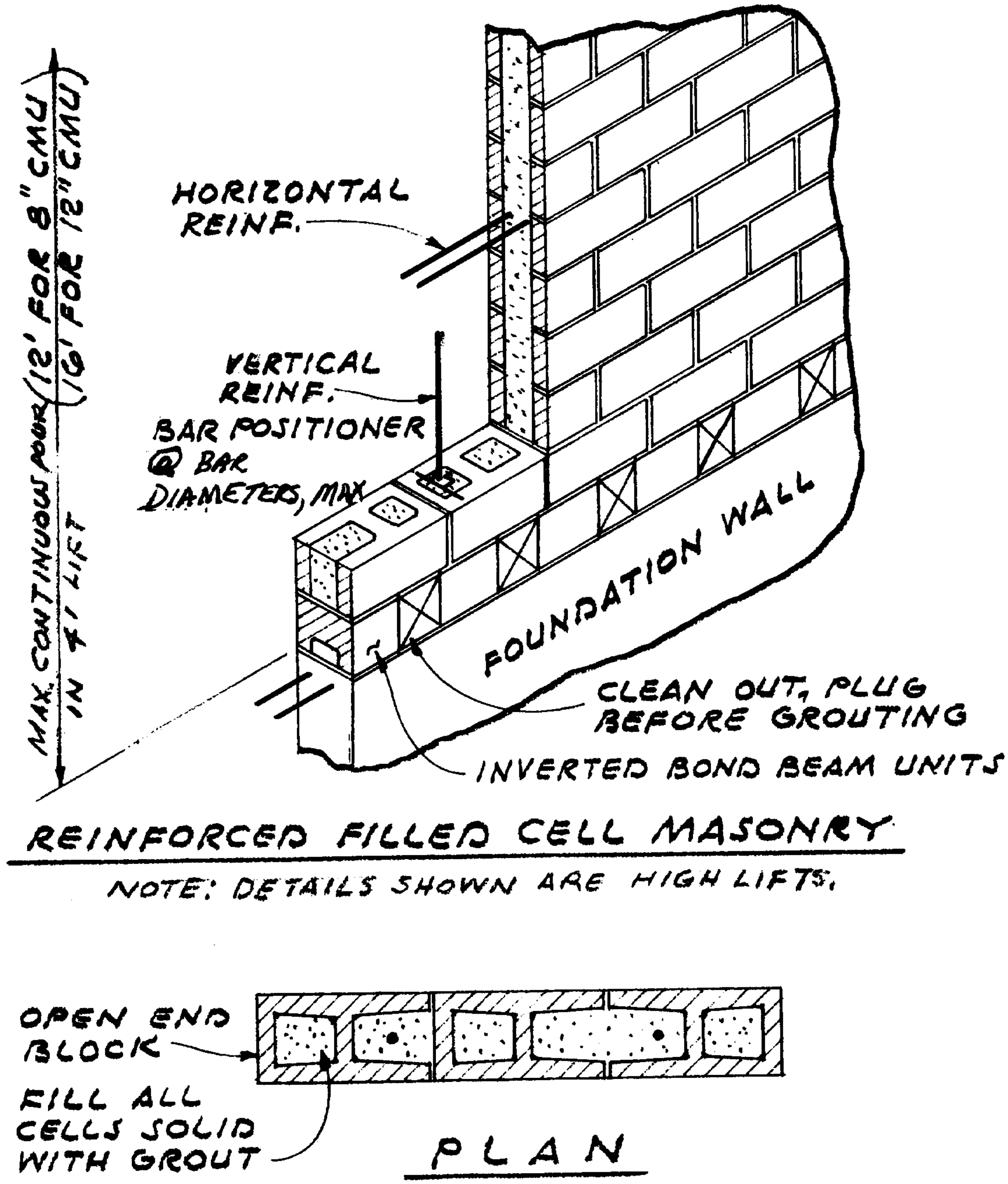


Figure 6-12. Reinforced filled-cell masonry.

masonry units and the average weight of completed walls.

(2) *Shearing stresses in hollow masonry.* Refer to TM 5-809-3/AFM 88-3, Chap 3 for the assumed effective area for hollow masonry and the equivalent thickness of hollow masonry for computing stress due to shear parallel to the face.

(3) *Boundary elements.* When masonry shear walls are used as part of a dual system (i.e., systems D1c and Did in SEAOC Table 1-G), special vertical boundary elements are required. These elements will be composed of structural steel or reinforced concrete in accordance with ACI 21.5.3.

h. Reinforcing. Typical reinforcement is shown in figure 6-14.

(1) *Minimum reinforcing.* Unit masonry must be reinforced not only for structural strength but to provide ductile properties and to hold it together in the event of severe seismic disturbance. All walls and partitions will be reinforced as required by structural calculations, but in no case with less than the minimum area of steel and the maximum spacing of bars prescribed below. The minimum reinforce-

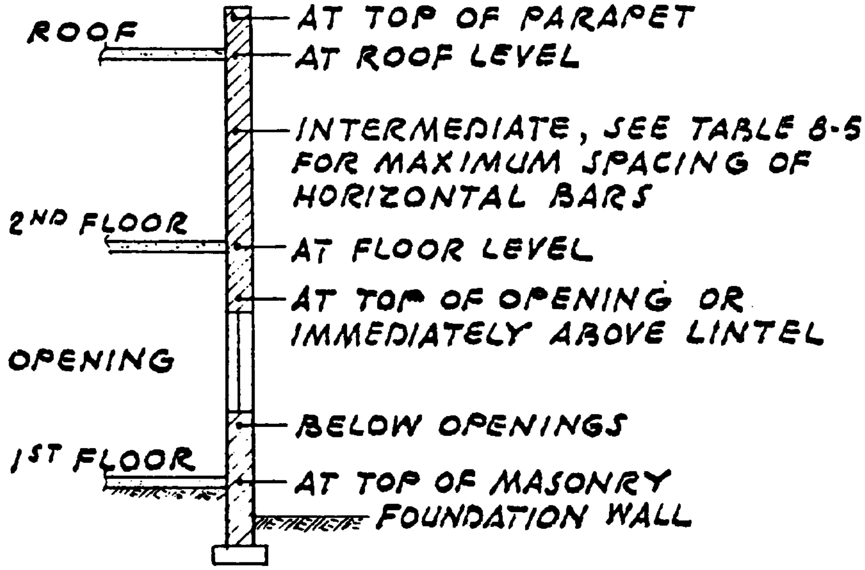


Figure 6-13. Location of bond beams.

ment and the maximum spacing of bars is controlled by the type of wall and the seismic zone. Table 6-1 applies. Only reinforcement that is continuous in any wall panel will be considered in computing the minimum area of reinforcement. Joint reinforcement used for crack control or mechanical bonding may be considered as part of the total minimum horizontal reinforcement but will not be used to resist computed stresses. (For Zone 1 structures, the exception for wall reinforcement under table 6-1 applies. Where the exception applies, masonry construction will conform to TM 5-809-3/AFM 88-3, Chap 3. Further additional bars will be provided around openings, at corners, at anchored intersections in wall piers, and at ends of wall panels as prescribed elsewhere in this chapter. Vertical bars in walls will be spliced as prescribed in TM 5-809-3/AFM 88-3, Chap 3.

(2) *Reinforcing in shear walls.* In Zones 3 and 4, reinforcement required to resist in-plane shear will be terminated with a standard hook or with an extension of proper embedment length beyond the reinforcing at the end of the wall section. The hook or extension may be turned up, down, or horizontally. Provisions will be made not to obstruct grout placement. Wall reinforcement terminating in columns or beams will be fully anchored into these elements.

(3) *Reinforcing in wall piers.* Horizontal reinforcement will be in the form of ties as shown in figure 6-14.

(4) *Column ties.* In Zones 3 and 4, the spacing of column ties will not be more than: 8 inches the full height for columns stressed by tensile or compressive axial overturning forces due to the seismic loads of chapter 3; 8 inches for the tops and bottoms of all other columns for a distance of one-sixth of the clear column height, but not less than 18 inches nor the maximum column dimension. Tie spacing for the remaining column height will be not more than 16 bar diameters, 48 tie diameters, or the least column dimension, but not more than 18 inches. Hooks in column ties will have a minimum turn of 135 degrees plus an extension of at least six bar diameters, but not less than 4 inches at the free end of the bar, except that where the ties are placed in the horizontal bed joints, the hook will consist of a 90-degree bend having a radius of not less than four bar diameters plus an extension of 32 bar diameters.

(5) *Reinforcing in stacked bond.* In Seismic Zone 2, the minimum horizontal reinforcement ratio shall be .0007 bt. This ratio shall be satisfied by uniformly distributed joint reinforcement fully embedded in mortar or by horizontal reinforcement spaced not over 4 feet and fully embedded in grout. In Seismic Zones 3 and 4 the minimum horizontal reinforcement ratio shall be .015 bt. If open end units are used and grouted solid, then the minimum horizontal reinforcement ratio shall be .0007 bt.

(6) *Reinforcing at wall openings.* Since the area around wall openings is vulnerable to failure, supplemental reinforcement is prescribed herein. For purposes of this paragraph, the term *jamb bars* will means bars of the same size, number, extent, and anchorages as the typical vertical stud reinforcement in that wall, and in no case less than one bar, #4 or larger. Refer to figure 6-15.

(a) Case I. Case I applies to all openings in nonstructural partitions over 100 square inches,

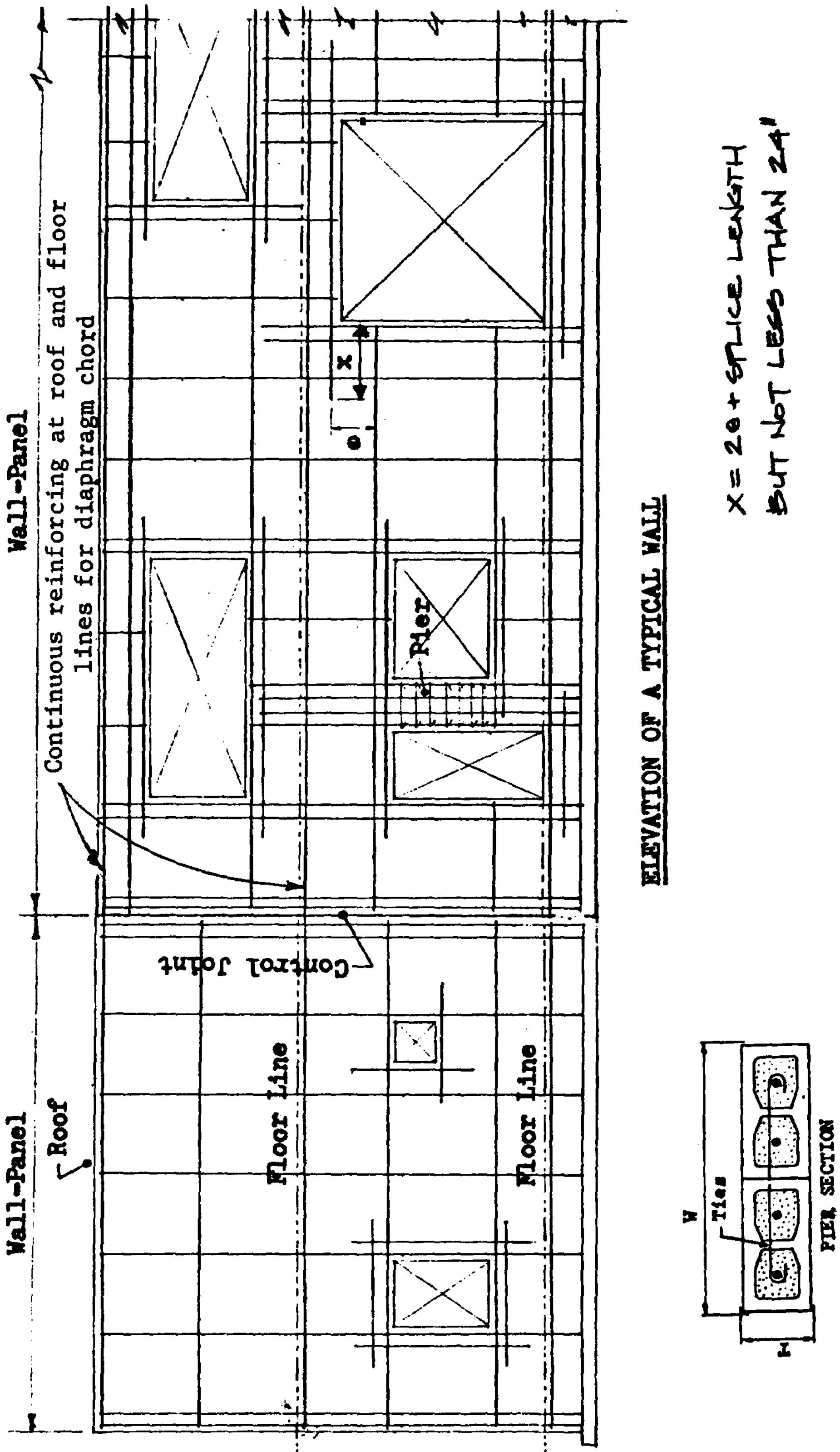


Figure 6-14. Typical wall reinforcement—reinforced masonry.

	Total Minimum Reinforcement (percent)[1,2]			Maximum spacing of bars (inches)					
				Vertical bars			Horizontal bars		
	Seismic Zone			Seismic Zone			Seismic Zone		
	4&3	2	1[3]	4&3	2	1[3]	4&3	2	1[3]
Structural	0.20	0.20	0.15	24	36	60	48	60	72
Nonstructural	0.15	0.15	0.15	48	60	72	84	84	96

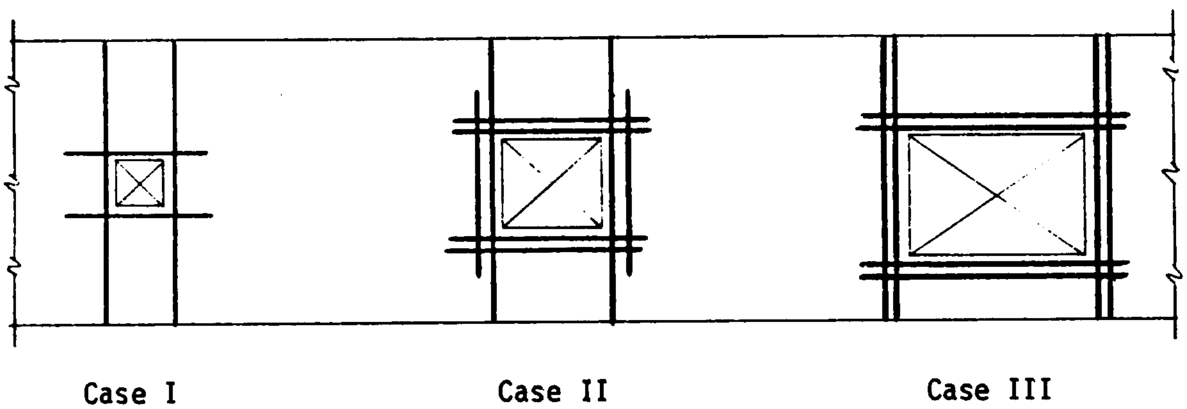
NOTES

[1]The total minimum reinforcement is the sum of the vertical and horizontal reinforcement; not less than 1/3 of the prescribed total minimum reinforcement will be used in either direction.

[2]The percentage of area reinforcement is based on gross area of wall (nominal dimensions).

[3]Exception: In seismic zone 1, one-story structures with eave heights not exceeding 14 feet; and two-story and three-story structures with story heights not exceeding 12 feet, the maximum spacing of vertical reinforcement in structural and nonstructural walls will be 6 feet and 8 feet, respectively. Vertical reinforcement will also be provided at each side of each opening and each corner. Horizontal reinforcement will be provided at top of footings and at the bottom and top of openings. These walls must be capable of resisting seismic zone 1 loads.

Table 6-1. Minimum wall reinforcement.



Refer to paragraph 6-9h (2) for application of Cases I, II, and III.

Figure 6-15. Reinforcement around wall openings.

and any opening in structural partitions or exterior walls that is 2 feet or less both ways but over 100 square inches. Jamb bars will be provided on each side of the opening *and* at least one bar, #4 or larger, will be provided at top and bottom of the opening. The lintel bars above the opening may serve as the top horizontal bar, and a bond beam bar at the bottom of the opening may serve as the bottom horizontal bar.

(b) Case II. Case II applies to exterior walls and structural partitions for any opening that exceeds 2 feet but is not over 4 feet in any direction. The perimeter reinforcement will be the same as in Case I *plus* additional reinforcement as follows: at least one bar, #4 or larger, will be provided on all four sides of the opening in addition to the bars required in Case I and shall extend not less than 40 bar diameters or 24 inches, whichever is larger, beyond the corners of the opening.

(c) Case III. Case III applies to any opening that exceeds 4 feet in either direction in exterior walls or structural partitions. The perimeter reinforcement will be the same as in Case II, except that vertical jamb bars will be provided in lieu of the

shorter vertical bars.

i. Additional details. See figure 6-16.

6-10. Wood stud shear wails.

a. General design criteria. The criteria used to design wood stud shear walls are presented in SEAOC, chapter 5 and the additional criteria in this section.

b. Allowable shears for plywood. Details of plywood sheathed walls are shown in figure 6-17, and the allowable shears are shown in figure 6-18. The usual one-third increase for short-term seismic loads is not applicable to these allowable shear values. When a combination of plywood and other materials is used, the shear strength of the walls will be determined by the values permitted for plywood alone.

c. Allowable shears for sheathing other than plywood. Figure 6-19 gives in tabular form the maximum height-width ratios and allowable shear per lineal foot for wood stud shear walls with various types of sheathing or plaster except for plywood sheathed walls. The usual one-third increase for short-time seismic loads is not applicable to these allowable shear values. The strength of any wood stud shear wall may be made up of a combination of the materials listed. In no case shall the allowable shears for combinations of materials exceed 600 pounds per lineal foot.

d. Deflections. Procedures for calculating the deflection of wood frame shear walls are not yet available. The maximum height-width limitations given herein are presumed to satisfactorily control deflections. Relative stiffnesses of wood stud shear walls will be measured by the effective lineal width of walls or piers between openings.

e. Let-in brace. Except when used in combination with diagonal sheathing or plywood, a 1-inch by 4-inch brace let into the studs may be used to resist an additional horizontal force not exceeding 1,000 pounds, provided the total value of the shear wall does not exceed 600 pounds per foot. Each such brace shall be nailed to each stud and to the top and bottom plates with two 8d nails.

f. Wall tie-down. The end studs of any plywood sheathed shear wall and/or shear wall pier will be tied down in such a manner as to resist the overturning forces produced by seismic forces parallel to the shear wall. This overturning force is sometimes of sufficient magnitude to require special steel attachment details. A commonly used detail is shown on figure 6-20. Tie-downs will be computed using the required stresses for wood and its fastenings increased one-third for seismic forces.

6-11. Steel stud shear walls.

a. Description of system. Steel studs may be used in lieu of wood studs in structural bearing walls. To function as shear walls, steel-stud walls need bracing. In principle, plywood sheathing could be used, but there are no available allowable shear values. Instead, it is customary to use diagonal braces made of steel straps welded to the face of the steel studs. Sheathing such as plywood or gypsum board may be used to serve architectural purposes such as containing insulation and backing up finishes.

b. Design criteria. This is system A3 in SEAOC Table 1-G: the R_w-value is 4. In this manual this category envisions structures with lateral forces that come from a roof and the upper part of walls, and that are small in magnitude. The system is limited to buildings of one story. The system will not be used for lateral support of masonry or precast panels.

c. Detailed design requirements. Figure 6-21 shows a few typical details for this light construction. Figure 6-21, sheet 1, shows a steel-strap brace that can be used to resist a maximum load of 1,000 pounds per brace. Note several essential features of that detail:

(1) The end of the diagonal strap is concentric with the stud and track and is welded all around to those two members.

(2) The vertical component of force is transmitted into the base via a structural steel angle whose upstanding leg is welded to the stud at locations very close to the flanges of the stud. This angle detail holds the base of the stud and prevents stresses and deflections that would occur in the track if the stud were connected solely to the track with the anchor bolt at some distance away.

(3) At least two straps are needed in each wall, arranged so that there is sufficient tension capacity in both directions of design force.

(4) The bottom track cannot be used to resist uplift by bending of the track web (as noted in the paragraph above concerning the brace detail).

(5) Both flanges of all studs must be braced against torsional buckling.

(6) Screws may not be used to resist forces in pullout.

(7) Provisions must be made for pretensioning the straps or otherwise ensuring that they are not loose.

(8) There will be some sheathing that will provide additional stiffening and some redundancy; if there is no sheathing, the designer should provide remedial solutions.

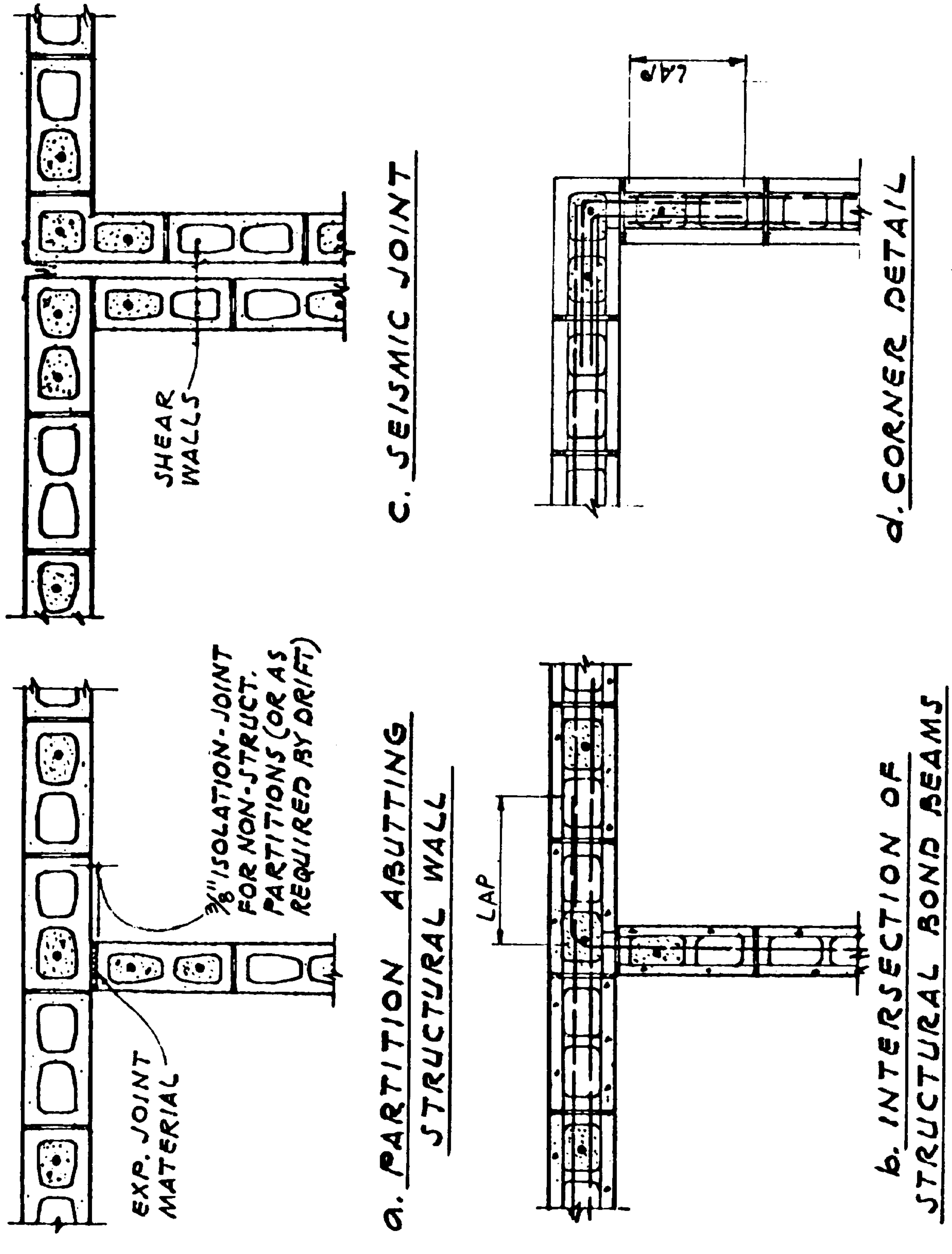


Figure 6-16. Masonry wall details.

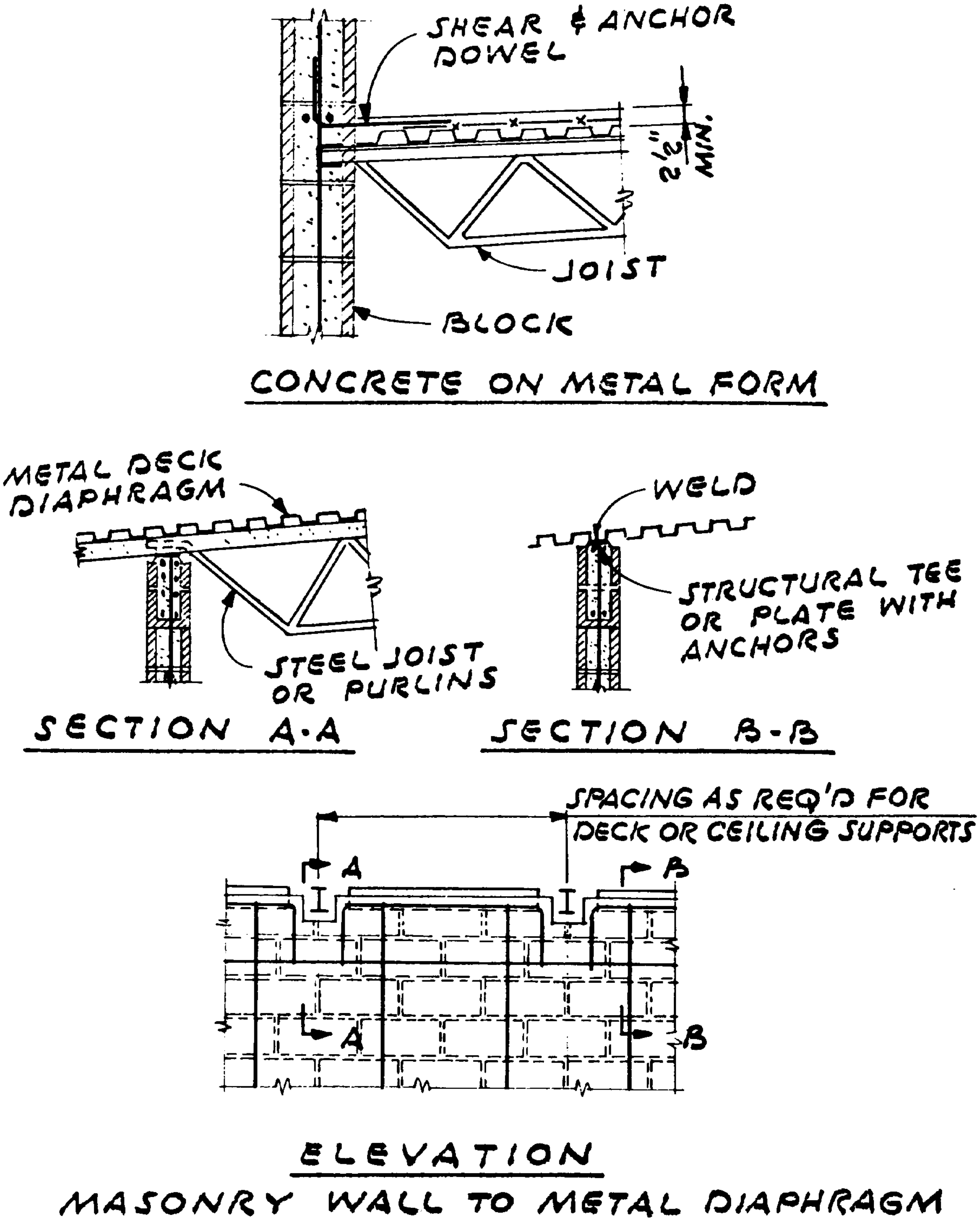
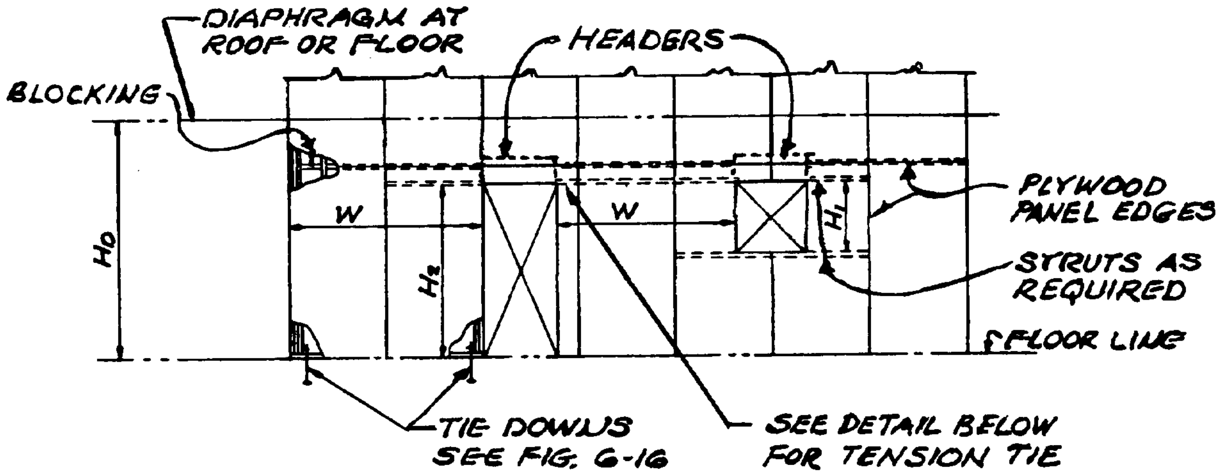
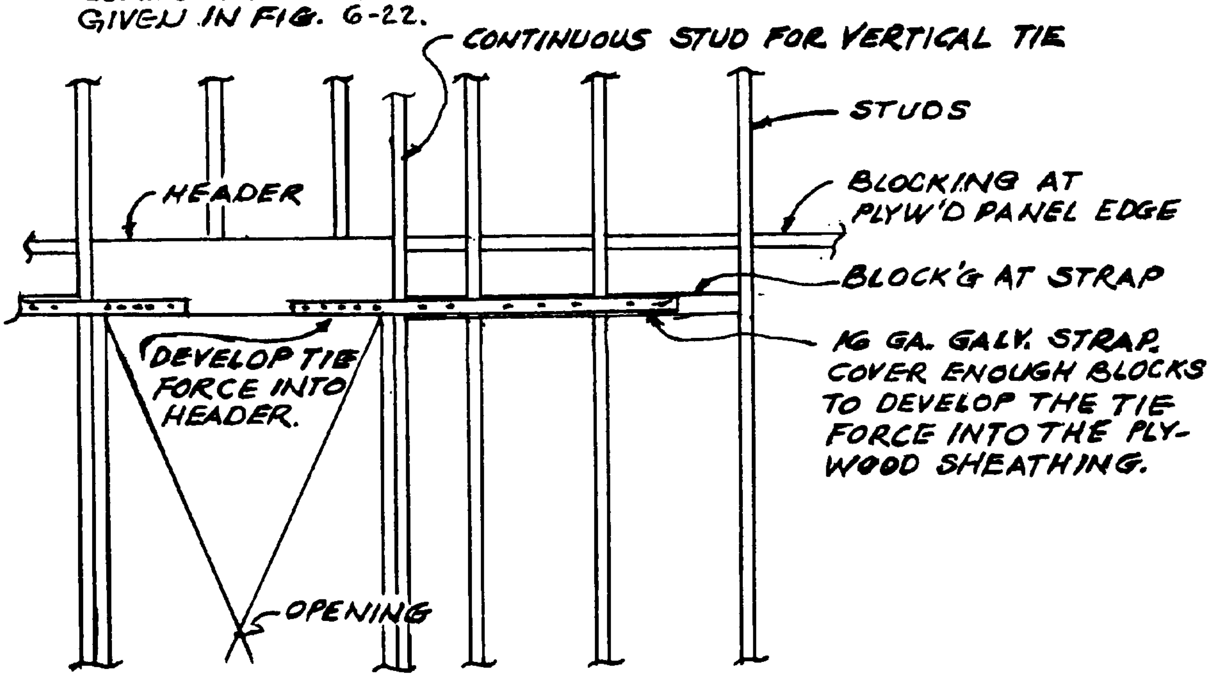


Figure 6-16. Continued.



NOTES:

1. FOR VALUES OF PLYWOOD SHEATHED SHEAR WALLS, SEE FIG. 6-15.
2. HEIGHT-WIDTH RATIO OF PLYWOOD SHEATHED SHEAR WALLS WILL BE LIMITED TO 3½ TO 1. $H_o/_W$ WILL BE USED FOR THE HEIGHT-WIDTH RATIO UNLESS STRUTS ARE DEVELOPED AT THE TOP AND BOTTOM OF THE OPENINGS IN WHICH CASE $H_1/_W$ OR $H_2/_W$ MAY BE USED.
3. THESE SHEAR WALLS SHALL NOT BE USED TO RESIST FORCES DUE TO CONCRETE NOR MASONRY MASSES, EXCEPT FOR VENEERS, SHOWER STALLS AND MINOR CONCRETE FILLS SUCH AS EQUIPMENT BASES.
4. THE USUAL 33⅓% INCREASE FOR SHORT-TIME SEISMIC LOADS IS NOT APPLICABLE TO THE ALLOWABLE SHEAR VALUES GIVEN IN FIG. 6-22.



DETAIL OF TENSION TIE - STRUT

Figure 6-17. Plywood-sheathed wood stud shear walls.

Recommended Shear (pounds per foot) for APA Panel Shear Walls with Framing of Douglas-Fir, Larch, or Southern Pine[a] for Wind or Seismic Loading[b]

Panel Grade	Minimum Nominal Panel Thickness (in.)	Minimum Nail Penetration in Framing (in.)	Panels Applied Direct to Framing: Nail Size (common or galvanized box)	Nail Spacing at Panel Edges (in.)				Panels Applied Over 1/2" or 5/8" Gypsum Sheathing: Nail Size (common or galvanized box)	Nail Spacing at Panel Edges (in.)			
				6	4	3	2[e]		6	4	3	2[e]
APA STRUCTURAL I grades	5/16	1-1/4	6d	200	300	390	510	8d	200	300	390	510
	3/8	1-1/2	8d	230[d]	360[d]	460[d]	610[d]	10d[f]	280	430	550	730
	7/16			255[d]	395[d]	505[d]	670[d]		—	—	—	—
	15/32			280	430	550	730		—	—	—	—
	15/32	1-5/8	10d[f]	340	510	665	870	—	—	—	—	—
APA RATED SHEATHING; APA RATED SIDING[g] and other APA grades except species Group 5.	5/16 or 1/4[c]	1-1/4	6d	180	270	350	450	8d	180	270	350	450
	3/8			200	300	390	510		200	300	390	510
	3/8	1-1/2	8d	220[d]	320[d]	410[d]	530[d]	10d[f]	260	380	490	640
	7/16			240[d]	350[d]	450[d]	585[d]		—	—	—	—
	15/32			260	380	490	640		—	—	—	—
	15/32	1-5/8	10d[f]	310	460	600	770	—	—	—	—	—
	19/32			340	510	665	870	—	—	—	—	—
APA RATED SIDING[g] and other APA grades except species Group 5	5/16[c]	1-1/4	Nail Size (galvanized casing) 6d	140	210	275	360	Nail Size (galvanized casing) 8d	140	210	275	360
	3/8	1-1/2	8d	160	240	310	410	10d[f]	160	240	310	410

(a) For framing of other species: (1) Find species group of lumber in the NFPA National Design Spec. (2)(a) For common or galvanized box nails, find shear value from table above for nail size for STRUCTURAL I panels (regardless of actual grade). (b) For galvanized casing nails, take shear value directly from table above. (3) Multiply this value by 0.82 for Lumber Group III or 0.65 for Lumber Group IV.

(b) All panel edges backed with 2-inch nominal or wider framing. Install panels either horizontally or vertically. Space nails 6 inches oc along intermediate framing members for 3/8-inch and 7/16-inch panels installed on studs spaced 24 inches oc. For other conditions and panel thicknesses, space nails 12 inches oc on intermediate supports.

(c) 3/8-inch or APA RATED SIDING - 16 oc is minimum recommended when applied direct to framing as exterior siding

(d) Shears may be increased to values shown for 15/32-inch sheathing with same nailing provided (1) studs are spaced a maximum of 16 inches oc, or (2) if panels are applied with long dimension across studs.

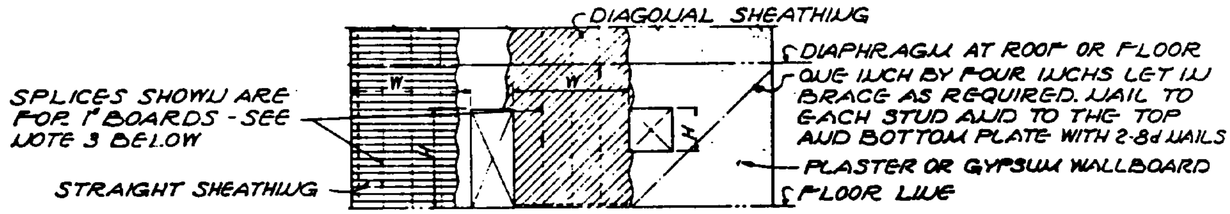
(e) Framing at adjoining panel edges shall be 3-inch nominal or wider, and nails shall be staggered where nails are spaced 2 inches oc.

(f) Framing at adjoining panel edges shall be 3-inch nominal or wider, and nails shall be staggered where 10d nails having penetration into framing of more than 1-5/8 inches are spaced 3 inches o.c.

(g) Values apply to all-veneer plywood APA RATED SIDING panels only. APA RATED SIDING - 16 oc plywood may be 11/32-inch, 3/8-inch or thicker. Thickness at point of nailing on panel edges governs shear values.

Typical Layout for Shear Walls

Figure 6-18. Allowable stresses for plywood-sheathed wood stud walls.



TYPICAL SHEAR WALL ELEVATION

VERTICAL SHEAR WALLS ONE SIDE ONLY	NAILING @ EACH BEARING COMMON UNLESS NOTED	MAXIMUM PIER HEIGHT-WIDTH RATIOS (H/w)	ALLOWABLE SHEAR LBS/LIN. FT.
1" STRAIGHT SHEATHING	2 - 8d	2:1	50
2" STRAIGHT SHEATHING	3 - 16d		40
CONVENTIONAL 1" DIAGONAL SHEATHING 1"x6"	2-8d(3-8d AT BOUNDARIES)	2:1	300
CONVENTIONAL 1" DIAGONAL SHEATHING 1"x8"	3-8d(4-8d AT BOUNDARIES)		
SPECIAL DIAGONAL SHEATHING	3-16d	3½:1	600
LATH & GYPSUM PLASTER	1¼" NO. 13 GAUGE 19/64" DIAMETER HEAD BLUED NAIL @ 5" o.c.	2:1	100
METAL LATH & PORTLAND CEMENT PLASTER	4d BLUED BOX NAILS @ 6" o.c. OR 1½" NO. 11 GAUGE 7/16" DIA. HEAD BARBED NAILS @ 6" o.c.	2:1	200
GYPSUM LATH, PLAIN OR PERFORATED 3/8" WITHOUT BLOCKING & ½" PLASTER	1⅛" NO. 13 GAUGE 19/64" DIAMETER HEAD BLUED NAIL @ 5" o.c.	1½:1	100
GYPSUM SHEATHING BOARD ½"x2'x8' WITHOUT BLOCKING	1¾" NO. 11 GAUGE 7/16" DIAMETER HEAD DIAMOND POINT GALVANIZED @ 4" o.c.	1½:1	75
GYPSUM SHEATHING BOARD ½"x4' WITH BLOCKING	1¾" NO. 11 GAUGE 7/16" DIAMETER HEAD DIAMOND POINT GALVANIZED @ 4" o.c.	1½:1	175
GYPSUM WALLBOARD (DRYWALL) ½" WITHOUT BLOCKING	5d OR @ 7" o.c.	1½:1	100
	1¼"x.098 GA. @ 4" o.c.	1½:1	125
GYPSUM WALLBOARD (DRYWALL) ½" WITH BLOCKING	5d OR @ 7" o.c.	1½:1	125
	1¼"x.098 GA. @ 4" o.c.	1½:1	150
GYPSUM WALLBOARD (DRYWALL) 5/8" WITH BLOCKING	6d @ 4" o.c.	1½:1	175
GYPSUM WALLBOARD (DRYWALL) 5/8" TWO-PLY WITH BLOCKING	6d @ 9" o.c. BASE PLY 8d @ 7" o.c. FACE PLY	1½:1	250

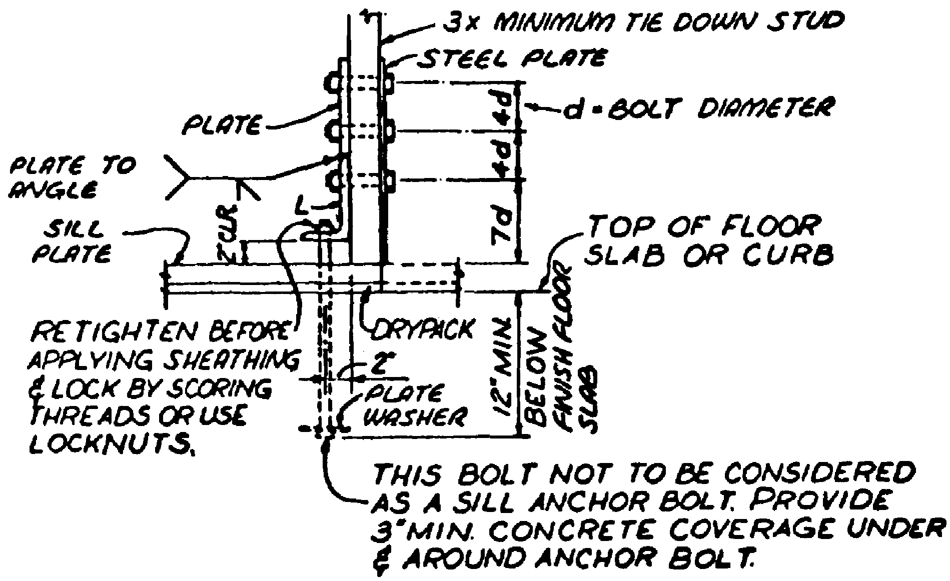
NOTES:

1. VALUES SHALL BE MODIFIED FOR PARTICULAR SPECIES OF WOOD IN ACCORDANCE WITH PERCENTAGES BELOW

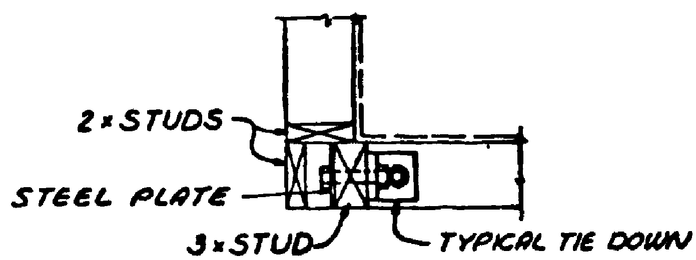
SPECIES	% OF TABULATED VALUES	SPECIES	% OF TABULATED VALUES	SPECIES	% OF TABULATED VALUES
DOUGLAS FIR (WEST COAST & INLAND)	100%	SPRUCE (SITKA)	75%	PINE (PONDEROSA)	65%
LARCH	100%	FIR (WHITE)	70%	PINE (SUGAR)	65%
HEMLOCK (WESTERN)	85%	CEDAR (WESTERN RED)	65%	PINE (LODGEPOLE)	65%
REDWOOD	80%	PINE (IDAHO WHITE)	65%	SPRUCE (ENGLEMANN)	65%

2. IF USED UNDER CONDITIONS OTHER THAN CONTINUOUSLY DRY, VALUES FOR WOOD SHEAR WALLS SHALL BE REDUCED TO 67% OF THE TABULATED VALUES.
3. DIAGONAL OR STRAIGHT SHEATHING - END JOINTS OF ADJACENT BOARDS WILL BE SEPARATED BY AT LEAST TWO JOIST OR RAFTER SPACES WITH AT LEAST TWO BOARDS BETWEEN JOINTS ON SAME SUPPORT.
4. SPECIAL DIAGONAL SHEATHING SHALL CONSIST OF TWO LAYERS OF 1" CONVENTIONAL DIAGONAL SHEATHING AT 90° TO EACH OTHER AND ON THE SAME FACE OF STUDS.
5. TYPE OF NAILS, SEE APPLICABLE AGENCY GUIDE SPECIFICATIONS.
6. THESE SHEAR WALLS SHALL NOT BE USED TO RESIST FORCES DUE TO CONCRETE NOR MASONRY MASSES, EXCEPT FOR VENEERS, SHOWER STALLS AND MINOR CONCRETE FILLS SUCH AS EQUIPMENT PADS.
7. THE USUAL 33⅓% INCREASE FOR SHORT-TIME SEISMIC LOADS IS NOT APPLICABLE TO THESE ALLOWABLE SHEAR VALUES.

Figure 6-19. Typical wood stud shear walls of various materials other than plywood.



TYPICAL TIE-DOWN DETAIL A



PLAN B

CORNER DETAIL AT TIE-DOWN

NOTE: Angle, bolts, plates, posts, footings, etc., to be designed for uplift.

Figure 6-20. Wood stud walls—typical tie-down details.

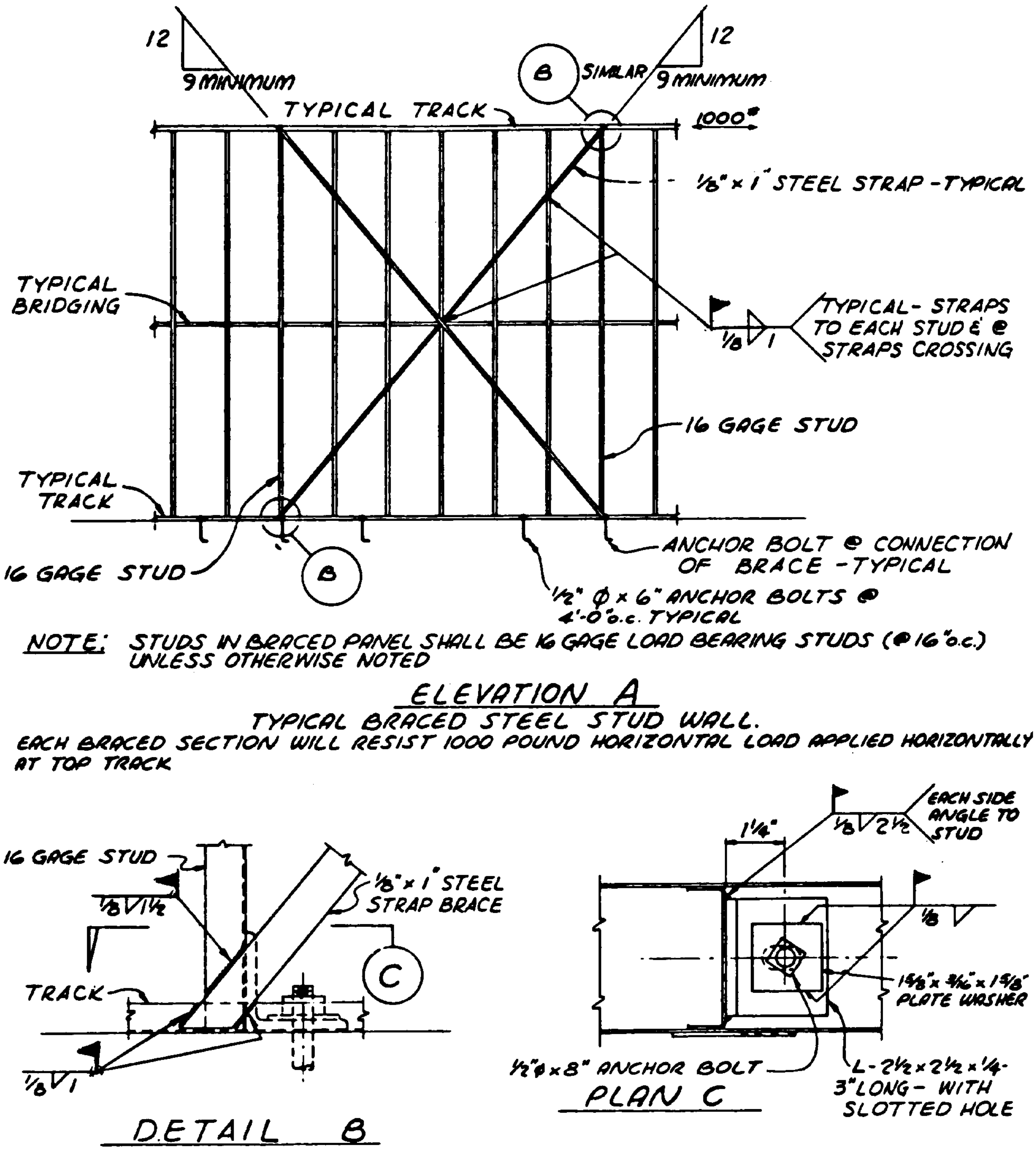


Figure 6-21. Steel stud shear walls—typical details.

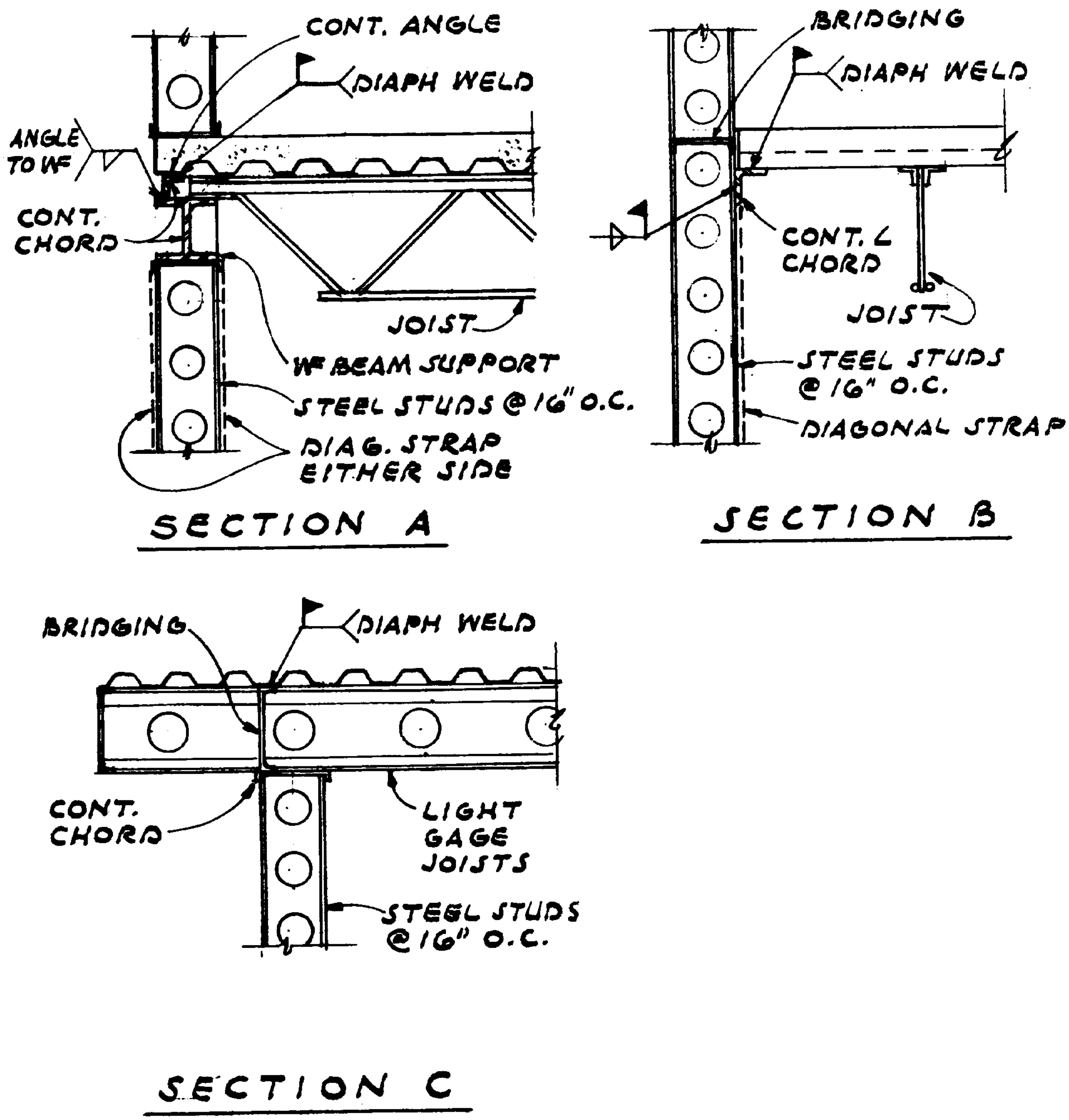


Figure 6-21. Continued.